APPENDIX D - GEOTECHNICAL

Appendix D
Geotechnical

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GeoEngineers

DRAFT

TECHNICAL MEMORANDUM NO. 13

Geotechnical, Geoenvironmental and Geologic Services

SUBJECT: **PRELIMINARY DESIGN REPORT GEOTECHNICAL ENGINEERING SERVICES PROPOSED DIVERSION DAM REPLACEMENT WHITE RIVER HYDROELECTRIC PROJECT BUCKLEY, WASHINGTON**

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APPROVED BY: Gordon M. Denby - GeoEngineers, Inc.

PROJECT NAME: White River Diversion Dam
PROJECT NO. GEI File No. 0186-115-R06
DATE: May 4, 1992

INTRODUCTION

This preliminary report presents the results of our geotechnical engineering services completed for the proposed diversion dam replacement for the White River Hydroelectric Project. The site is located on the White River approximately 1/2 mile east of the SR 410 highway near Buckley, Washington as shown on the Vicinity Map and Site Plan, Figure 1.

The conclusions and recommendations presented herein supersede those in our interim report dated January 22, 1991. The January 22 interim report was prepared for use by HDR in the conceptual design phase of the project. This preliminary design report includes an updated description of the project and the results of additional explorations completed for the project. It also includes preliminary geotechnical engineering conclusions and recommendations together with a section which addresses, on a preliminary basis, construction considerations. A final design report which will include more detailed geotechnical recommendations and construction considerations for use by HDR Engineering, Inc. in developing final design documents will be prepared during final design efforts based on additional geotechnical studies. A section outlining additional geotechnical services recommended for the final design phase is presented in a section at the end of this preliminary report.

Our understanding of the project is based on discussions with Mr. Bob King of HDR and Messrs. Mike Blanchette and Wayne Porter of Puget (Puget Sound Power and Light Company), draft plans by HDR entitled "P.S.P.& L. Co., Application for License, White River Hydroelectric Project, FERC Project No. 2494, Drawings 1 through 17" (undated), transmitted April 8, 1992, and our experience with other work for Puget at the White River Hydroelectric Project.

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PROJECT DESCRIPTION

The White River Project is an existing hydroelectric facility which consists of a diversion dam and intake structure; an 8-mile-long series of flumes, canals, and basins; a set of fish screens; a storage reservoir (Lake Tapps); an intake tunnel; a forebay well; four penstocks; and a single four-unit powerhouse. It began operation in 1911 and is currently rated at 63.4 MW. The diversion dam is located approximately one mile north-northeast of Buckley, Washington.

The existing diversion dam consists of a 352-foot-long by approximately 4-foot-thick concrete and rock-filled timber crib structure. Wood flashboards extend 7 feet above the crib structure. The water level behind the dam is maintained at Elevation 671 feet. To facilitate flashboard replacement and removal, a cable tramway is suspended over the dam. The dam is protected on both upstream and downstream faces by timber aprons. Six-foot-deep concrete cutoff walls extending about 9.5 feet below original riverbed underlie the length of the dam. These cutoffs are located near the upstream and downstream edges of the dam. The dam abutments consist of thick unreinforced concrete wing walls.

The proposed project consists of replacing the existing timber crib dam. The northern (right two-thirds of the existing dam foundation will remain in place and be used as foundation support for the replacement dam. The replacement dam will consist of two radial gates (16- and 35-foot-wide), two 50-foot-wide rubber weirs, and six 20-foot-wide removable, fixed crest, concrete panels. The replacement dam will have provisions to regulate flows, allow for fisheries flow requirements, and prevent bedload sediment from entering the intake. As part of the project, the existing intake and the headgates will be modified. A maintenance building, control building and garage /shop building will be constructed in the vicinity of the intake.

Construction of the replacement dam will be completed in two phases. Phase I includes the fixed crest, concrete panels and the rubber weirs. Phase II includes the two radial gates and the modification to the intake and headgates. Phase I requires construction of a 250-foot-long cofferdam upstream of the existing dam consisting primarily of a sand and gravel berm. The south end of this cofferdam, which is near the main channel of the river, will consist of either crushed rock protected by riprap or sheet pile cells. Phase II construction will require a 200-footlong, sheet pile cellular cofferdam upstream and a 100-foot-long earth cofferdam downstream of the dam.

Modifications to the existing dike and access road along the north shore of the river upstream of the dam are planned to reduce the potential for flooding near the existing fish hatchery. From the right abutment to about 800 feet upstream of the dam, the existing access road will be heightened by 2 to 5 feet. From 800 to about 1,300 feet upstream of the abutment, the existing shoreline dike will be heightened by about 3 to 5 feet.

Others have completed several studies for previous projects at and near the dam site. Aerial photographs and historical construction photos and records are also available. A list of references reviewed for this project is summarized at the end of the text.

SCOPE

The purpose of our services is to provide geotechnical design information to HDR for use in developing various design alternatives for the proposed diversion dam replacement. Our specific scope of services includes:

1. Review Available Data
   - Review existing project information made available from Puget Power's files.
   - Review subsurface and ground water information in our files.
   - Review historic aerial photographs to evaluate migration of the river.
2. Exploration Program
   - Drill two 50-foot-deep borings and two 40-foot-deep borings along the diversion dam alignment for use in foundation design of the proposed replacement dam.
   - Install one 2-inch-diameter well to 25 feet in one boring for conducting a slug (permeability) test.
   - Excavate eight shallow test pits along the existing north dike to obtain information for stability.
   - Excavate several test pits in the riverbed downstream of the existing dam to identify the nature of near-surface river deposits. This work was completed after the January 22 interim report was transmitted.
3. Field Reconnaissance
   - Complete a field reconnaissance to evaluate the extent and condition of existing channel erosion within about 'h mile upstream of the dam.
   - Complete a field reconnaissance along the north bank of the White River to evaluate potential locations for the construction of the flood protection dike in this area.
4. Field Data Interpretation
   - Review and interpret boring logs, test pit logs, laboratory data, and results from the geophysical survey.
   - Develop soil cross sections along the dam and north dike, as appropriate.
   - Prepare final logs of the explorations and a site plan showing boring, test pit and geophysical line locations.
5. Foundation Support
   - Develop foundation design recommendations for the support of the proposed diversion dam structure, including use of the existing dam foundation in new dam construction.
   - Provide estimates of allowable soil-bearing pressure.
   - Provide estimates of allowable soil friction values and earth. pressures.
   - Provide estimates of settlements for the proposed dam structure.

6. Channel Erosion Protection
   - Evaluate areas along the river channel (within mile upstream of the dam) requiring erosion protection, based on site reconnaissance and historic channel migration.
   - Develop recommendations for channel stabilization within about 1/2 mile upstream of the dam. This will include recommendations for:
     - size and layout of training groins in the river
     - size, thickness and geometry of riprap protection
     - gradation requirements
     - filter material requirements, if needed
     - construction considerations
7. Construction Considerations
   - Provide geotechnical input and construction considerations for the proposed diversion dam, including:
     - cellular cofferdam construction
     - earth cofferdam construction
     - fill material selection and placement techniques
     - upstream slope protection
     - temporary dewatering
8. North Dike Stability
   - Evaluate subsurface soil conditions at and near the existing north dike upstream of the dam.
   - Perform stability analyses as appropriate for the dike.
   - Develop recommendations for remedial action, if necessary, to improve the effectiveness of the dike and its stability.
   - Prepare recommendations for the design and construction of the north dike, including fill material selection, placement techniques and slope protection.
9. Seepage Analysis
   - Estimate the rate and quantity of seepage flow below and around the existing dam using a flow net analysis method.
   - Develop estimates of uplift pressure on the base of the proposed concrete dam.
   - Evaluate the potential for piping to occur below or around the dam.
   - Estimate rate and quantity of seepage flow beneath the temporary earth and cellular cofferdams using a flow net analysis method.
10. Meetings
    - Attend an estimated five to six meetings with Puget and other team members to develop design concepts and to discuss our findings.
11. Progress Reports
    - Prepare two interim progress letters summarizing services completed by GEI.

12. Draft and Final Reports

- Prepare the draft preliminary report for review by team members summarizing results from our field and laboratory programs together with our conclusions and recommendations.
- Prepare 10 copies of a final preliminary report which incorporates review comments.

SITE CONDITIONS

GEOLOGIC HISTORY

The dam site is located in a geologic region characterized by thin alluvial deposits overlying Osceola Mudflow deposits up to 70 feet thick in some areas. The Osceola Mudflow blanketed an extensive portion of the eastern Puget Sound Lowland approximately 3,700 years ago. The mudflow originated on the north flank of Mount Rainier, flowed down the White River Valley, and covered a wide area including present-day Buckley with several tens of feet of sediment. The mudflow sediment typically consists of cobbly, silty sand and gravel with cobbles and occasional boulders. The mudflow occurred in a series of separate flows in between and after which alluvial soils consisting of sand and gravel with cobbles and boulders were deposited by the White River. Moreover, water flow over and through the surface layer of the exposed mudflow deposits probably washed silt material portions out of the soil. This process resulted in a relatively inhomogeneous mix of clean alluvial and mudflow deposits and siltier mudflow deposits in the upper 5 to 10 feet of soil. Glacially deposited sands and gravels typically underlie the mudflow deposits.

Soils exposed within a 25-foot-high riverbank along the south side of the White River, immediately upstream from the diversion dam, show approximately 10 feet of an alluvial deposit overlying Osceola Mudflow sediments, indicating that the river has and continues to incise through the mudflow deposits at this location.

SURFACE CONDITIONS

The White River Fish Hatchery is located on the north side of the river just upstream of the dam, and the intake structure and flume are located near the south abutment of the dam (refer to Figure 1). A gravel dike constructed in stages since 1911 is situated along the north bank of the White River to protect the bank from erosion and the fish hatchery from flooding during periods of high water. The approximate location of the north dike is outlined in Figure 1. The older portion of the dike is approximately 3 to 6 feet high (with respect to native ground surface north of the dike) and extends about 1,800 feet upstream of the dam. This portion of the dike is covered with brush and trees ranging in diameter (at breast height) from 4 to 36 inches. It is protected on the river side by large boulders and concrete rubble up to 8 feet in size. The newer portion of the dike is approximately 8 to 12 feet high (with respect to the ground surface north of the dike) and extends from about 1,800 to 2,500 feet upstream of the diversion dam. It was constructed in approximately 1968. This portion of the dike is composed of sand and gravel and is protected by quarry rock typically ranging from 2 to 24 inches in size. The top of the dike ranges in elevation from approximately 673 feet near the dam to approximately 689 feet 2,500 feet upstream of the dam. The water surface elevations of the White River near these locations were approximately 670 and 678 feet respectively on September 14, 1990 based on hand level measurements.

An access road leading to the right abutment of the dam was constructed in the late 1980s. The location of this access road is shown in Figure 1. The road appears to be composed of sand and gravel, based on evaluation of surface soils. It is protected on the river side by 12-inchdiameter quarry spalls. The road extends from the right abutment east approximately 800 feet.

The topography on the north side of the river (north of the existing dike) is relatively flat, except for the access road and fill pads for the hatchery, with elevations ranging from 665 feet near the fish hatchery to about 680 feet a half mile upstream of the diversion dam. Large sand and gravel bars are present near the north bank of the river, just upstream of the dam. The ground surface elevations of the sand bars typically range from 670 to 678 feet.

A 25-foot-high bluff forms the south bank of the river immediately upstream of the diversion dam. The ground surface south of the bluff is relatively flat and ranges in elevation from about 690 to 695 feet. The bluff extends approximately 900 feet upstream of the diversion dam. Further upstream, along the south side of the river, the ground surface is several feet above the river level. The topography in this area is relatively fiat with ground surface elevations ranging from 675 to 685 feet.

The majority of the site upstream of the diversion dam is heavily forested with fir, cedar, and alder trees. Low undergrowth consists of grasses, nettles, ferns, and scattered blackberry bushes. Scattered open areas of grass and brush with scattered alder and maple trees are present along the eastern end of the dike on the north side and on top of the bluff along the south side of the river. The hatchery site is an open grassy area.

A geotechnical engineer and an engineering geologist from our firm completed a field reconnaissance on September 18, 1990 to evaluate the existing condition of the north dike and riverbanks for a half-mile upstream of the dam. We completed several other site reconnaissances subsequently and reviewed aerial photographs of the area taken in 1936, 1968, 1980 and 1985. Additional historical information regarding construction of the dike along the north bank was obtained from discussions with Puget engineers and King County maintenance personnel.

The newer eastern portion of the dike constructed along the north bank and extending about 1,800 to 2,500 feet upstream of the dam appears to be relatively stable at this time. Some loss of finer rock appears to have occurred in the unmaintained areas of the dike. According to King County maintenance personnel, new rock protection was placed over the dike at the north and south ends where more significant loss of rock protection occurred during the 1989/90 winter high river flow. We understand that King County repairs the dike every few years on an as-needed basis. Repair of the dike protection involves dumping and spreading well-graded crushed quarry rock (obtained from the Enumclaw quarry located on S.E. 416th Street in Enumclaw, Washington) and spreading the material in a 1- to 3-foot lift. Maximum size of the quarry rock is about 2 feet.

The results of our reconnaissance and aerial photograph review indicate that relatively little erosion of the riverbanks for a half mile upstream of the dam has occurred since the diversion dam was constructed in 1911. From aerial photographs, it appears that the bluff along the south bank just upstream of the dam may have receded 5 to 10 feet since 1936. In other areas of the river channel, the banks appear to have aggraded since 1936. Sediment accumulation is particularly evident along the north side of the river just upstream of the dam. Sand and gravel bars on the order of 100 feet wide presently extend about 1000 feet upstream of the dam. Only portions of these bars are evident in the 1936 photograph. In general, the river channel appears to have become narrower and more stable (i.e., less meandering) since about 1968. During our field reconnaissances, we did not encounter evidence of any significant erosion along the banks. Some minor sloughing along the steep bluff on the south bank was observed. Our experience with similar bluffs composed of mudflow deposits elsewhere in the White River indicates that while the river tends to meander along the valley, very little lateral erosion of the bluffs occurs.

The sand and gravel bars near the north side of the river are currently covered with brush and small to moderate-sized trees. Hand holes and shallow erosion cuts along the river indicate that the near-surface soil conditions in these bars typically consist of 1/2 to 2 feet of fine to medium sand underlain by sandy gravel with cobbles in the western two-thirds of the bars. In the eastern one-third, the surface of the bars is predominately covered with gravel and cobbles ranging from 1 to 10 inches in size. Some sand is mixed in with the gravel and cobbles. Based on visual observations of the hand holes, we estimate the average percentage of cobbles in the upper 2 to 3 feet of the bars to be about 20 to 30 percent by volume.

The vegetation along the riverbank north of the sand and gravel bars is well established. The large rock and concrete bank protection placed on the older dike in the early 1900s is surrounded by brush and trees. It appears that this area of the bank has not been subject to significant water flow or erosion for many years. The size of trees along this existing dike ranges from 4 to over 36 inches in diameter. A summary of trees sizes along the dike alignment is presented in Table 1.

SUBSURFACE EXPLORATIONS

Subsurface soil and ground water conditions at the diversion dam were explored by drilling four borings to depths of 40 to 50 feet below the dam or ground surface, and by excavating five test pits downstream of the dam and digging four hand holes with a shovel. Borings B-1 and B-2 were completed in the White River by drilling through the wood and concrete apron on the downstream side of the diversion dam. Borings B-3 and B-4 were located near the north and south abutments of the dam, respectively. A 2-inch-diameter well was installed to 25 feet in boring B-3 for the purpose of conducting a slug (permeability) test.

Four test pits, TP-9 through TP-13, were completed in the riverbed downstream of the existing dam on August 2, 1991 using a track-mounted excavator. The purpose of these test pits was to evaluate the near-surface soils in the riverbed with respect to grain-size distribution. The test pits were excavated to depths 7 to 16 feet below the bottom of the riverbed.

Four hand holes were excavated in the riverbank and sand bars near the right abutment just upstream of the dam. The holes were excavated using a hand shovel to depths of 1 to 3 feet below the riverbed. The purpose of these holes was to evaluate the near-surface soils upstream of the dam for possible use as fill material during construction. The locations of these hand holes are shown in Figure 2.

A geophysical survey using vertical electrical soundings (VES) and over water seismic refraction was completed along the diversion dam alignment by Mr. Sig Schwarz to provide subsurface information between the borings. The geophysical survey report prepared by Mr. Schwarz is included in Appendix B.

Subsurface soil and ground water conditions along the existing north dike were explored by excavating 8 test pits, designated TP-1 through TP-8, through the dike.

The locations of our borings, test-pits, and the geophysical survey are shown in Figures 1 and 2. A description of the field explorations, lab testing procedures, and the logs of the explorations are presented in Appendix A. Specific subsurface conditions evaluated from the exploration program are discussed in the following sections.

SUBSURFACE SOIL CONDITIONS

Dam Alignment

A soil profile along the dam axis and interpreted from the borings and test pits is shown in Figure 3. The location of the profile is shown in Figure 2. The soils encountered in our borings and test pits indicate the diversion dam, the bottom of which is at Elevation 658 feet, is underlain by 3 to 9 feet (Elevation 655 to 649 feet) of loose to medium dense sandy gravel with occasional cobbles and boulders. The presence of a cobble or boulder resulted in refusal in one test pit (TP-10) at a depth of 12 feet. Above this level excavation of the soil was relatively easy indicating loose conditions. This material may be high energy alluvial deposits or Osceola Mudflow sediments with the finer grain-size particles washed out. Below 3 to 9 feet, Osceola Mudflow deposits consisting of medium dense to dense gravel typically with silt, sand, cobbles, and occasional boulders were encountered to a depth of 31 to 45 feet (Elevation 627 to 613 feet) below the base of the dam (top of subgrade below dam). The blow counts recorded during sampling indicate very dense soils; however, the presence of gravel, cobbles and boulders may have caused unrepresentatively high blow counts. Based on the rate of drilling, the soils typically appear to be medium dense to dense. Also, some of our samples appeared to be slightly washed by the drilling and sampling procedures indicating the in-place soil may have higher fines contents than were measured in the lab tests. Within the mudflow deposit, pockets of material ranging from gravelly, sandy silt to sandy gravels with no silt were also encountered. Underlying the mudflow deposit, dense, gravelly sands with occasional cobbles were encountered to the depth explored, 47 feet (Elevation 611 feet) below the base of the dam. These soils are either glacial recessional or alluvium deposits.

The soils encountered in our borings north and south of the existing abutments indicate about 10 feet (to Elevation 663 feet) of silty sand and sand and gravel (alluvial flood plain deposits) overlying Osceola silty sandy gravel mudflow deposits. The upper 6 to 7 feet (to Elevation 667 to 666 feet) of the soil in B4 at the north abutment may be recently placed fill.

The results from the geophysical survey appear to correlate well with the data obtained from the borings and test pits. From interpretation of the geophysical data, the dam alignment appears to be underlain by a thin mantle of relatively clean granular material which is underlain by siltier sand and gravel with pockets of cleaner material. The geophysical data also indicate that bedrock is deeper than 60 to 70 feet (Elevation 598 to 588 feet) in the dam site area. The soil profile interpreted from the geophysical data is shown in Appendix B.

North Dike

Test pits 1 through 6 were excavated along the newer section of dike, and test pits 7 and 8 were excavated along the older section of dike. All test pits encountered loose to medium dense gravelly sand with cobbles, boulders and varying amounts of silt to the depths explored. The dikes appear to have been constructed over the native sands and gravels (alluvium) using alluvial material. Therefore, it was difficult in some cases to distinguish between the dike material and the underlying native soils. The depth of dike fill ranges up to approximately 14 feet along the newer section and 3 to 6 feet along the older section.

HYDROGEOLOGIC CONDITIONS

Water level in borings B-2 and B-3 was encountered initially at the river level (Elevation 650 feet). In addition, artesian conditions were encountered at a depth of approximately 30 feet (Elevation 631 feet) in boring B-1 when the air-rotary casing drilled into a clean sandy gravel zone. The water level in the casing rose to approximately 8 feet above the downstream river level (to about Elevation 667 feet, which is about 3 feet below the upstream water level). The artesian conditions continued for the duration of the boring which was completed the following day. An estimate of the flow rate from this boring was from 5 to 10 gpm. No artesian conditions were encountered in any of the other borings.

The ground water level in boring B-4 was observed at a depth of 3 feet below the ground surface (Elevation 670 feet), which corresponds to about 1 foot above the upstream river level. The ground water level in the well installed at boring B-3 was recorded at approximately 12 feet below the ground surface (Elevation 661 feet), which is approximately 9 feet below the level of the river upstream of the diversion dam.

No ground water seepage was encountered in the test pits. However, slight to moderate caving was encountered in test pits 4 and 5 at the east end of the north bank.

The ground water levels observed in our explorations indicate a relatively complex ground water system. Along the north bank near the dam, boring B-3 showed ground water at elevation 661 or about 9 feet below the level of the river located about 25 feet away. Wells installed in other explorations completed by GeoEngineers for the hatchery showed ground water elevations between 660 and 665 feet at a distance of 300 to 600 feet from the north bank of the river (Sverdrup Corporation, 1967; refer to List of Reference Reports at end of text). This suggests that the regional ground water level on the north side is 7 to 10 feet lower than the river level upstream of the diversion dam. No water seeps were observed along the north bank. We interpret this information to imply that water from the river is discharging into the north bank from the river.

Seeps and wet zones of soil were observed along the south bank of the White River upstream and downstream of the diversion dam. The ground water level measured in boring B-4 located 20 to 30 feet south of the river is about 2 feet above the river level. This indicates ground water flow is north toward the river. Ground water recharge from the south bank could also account, in part, for the artesian conditions observed while drilling boring B-i located about 60 feet from the south bank. We expect that the reason no artesian conditions were observed while drilling boring B-2 is that the excess pressure head has dissipated at this distance (260 feet) from the south bank.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

In our opinion, the proposed diversion dam replacement may be supported on shallow foundations bearing on the undisturbed native materials or compacted structural fill. Provided the concrete foundation of the existing dam is structurally sound (to be determined by the structural engineer) and the absence of voids below the concrete foundation is demonstrated, we consider it is also feasible to support the replacement dam on the existing dam foundation, as planned. Because the radial gate section will be sensitive to differential settlements, it is possible that these structures may need to be pile-supported. Additional analyses during final design will be needed to determine the most appropriate foundation system for the radial gates.

Based on the differential head condition across the dam, we expect low to moderate seepage under and around the dam provided the sand and gravel deposit immediately below the base of the dam is suitably compacted and cutoffs of sufficient depth are constructed as part of the replacement dam.

The existing dam appears to be stable from a geotechnical standpoint. We found no evidence of significant lateral or vertical movement of the dam. Some loss of soil was observed below the downstream apron. We attribute this loss to erosion by turbulence and undercutting at the edge of the apron. No piping of subgrade soils, either below or around the dam, was observed.

The existing north dike and underlying native materials are competent. With proper design and construction, the dike may be raised up to 7 feet by adding new fill and riprap protection without significantly reducing existing dike stability. The existing access road also appears to be suitable for placement of additional fill for new dike construction. We present preliminary recommendations for raising the level of the dike in these two areas in subsequent sections.

Based on site reconnaissance and a review of air photographs it appears that relatively little erosion has occurred along the north and south banks of the White River within 1/2 mile upstream of the dam since 1936 (reference 1936 air photograph). Existing riprap protection of the north bank dike has adequately protected the dike fill with minor repair every few years.

The conclusions and recommendations in the following sections are specific to the existing diversion dam and river conditions and general foundation design input for the replacement dam. Recommendations regarding the replacement dam should, in general, be considered preliminary. Specific geotechnical recommendations and construction considerations pertaining to the proposed diversion replacement dam and related remedial measures will be prepared during the final design phase of the project after additional geotechnical information is obtained.

FOUNDATION SUPPORT

The dam may be supported directly on undisturbed or compacted native soils (either Osceola mudflow or alluvial deposits), compacted structural fill or the existing concrete foundation, provided it is structurally sound and there are no voids below the foundation. We recommend that a structural engineer verify the structural integrity of the existing dam structure during final design by means of concrete coring. The foundation materials underlying the existing dam foundation should be explored at the same time to confirm their adequacy for supporting the rubber weirs and fixed crest concrete panels. This is described in a later section.

Foundations placed on competent bearing strata may be designed for an allowable bearing pressure of 2,000 psf (pounds per square foot) for combined dead and long-term live loads. The allowable soil-bearing value may be increased by one-third for transient loads such as those induced by seismic events. This soil-bearing value is based on correlation between blow count and allowable soil-bearing values reported in the literature and our experience with similar soils. We estimate that the existing structure exerts an effective pressure of 600 to 1,000 psf on the subgrade soils. Higher allowable soil bearing pressures may be possible after further evaluation of settlements.

Settlement due to foundation loading is expected to be primarily elastic in nature and will occur essentially as the loads are applied. The magnitude of settlement will be dependent on the type of foundation and the actual design loads of the replacement dam. Preliminary analyses indicate that total elastic settlements on the order of 1 inch or less may occur under a design load of 2,000 psf. Total settlement under other loads would be directly proportional to the applied load. Differential settlements below a particular foundation may range from 1/8 to 1/2 of the total settlement, depending on soil variability, load eccentricity and foundation stiffness. Since there is no evidence of appreciable amounts of organics or fine-grained soil at the site, we anticipate that long-term settlement due to plastic deformation or creep of the soil will be negligible.

We understand the radial gates are sensitive to differential settlement. The preliminary settlement estimates presented above may exceed the allowable foundation settlements. Further analyses will be required during final design to refine these estimates. If, at that time, it is found that settlement of on-grade foundations may be excessive, alternate foundation systems will need to be considered for the radial gates. Other methods of foundation support include replacing a portion of the native soils with compacted import material such as crushed rock. Pile foundations bearing in the underlying mudflow deposits may also be feasible. These will be evaluated further, if appropriate, during final design.

LATERAL LOAD CONSIDERATIONS

Active earth pressures on the upstream dike walls or cutoffs may be evaluated using a triangular-shaped equivalent fluid earth pressure of 18 pcf (pounds per cubic foot) times the height of soil behind the dam wall or cutoff. Hydrostatic and dynamic water pressure should be added to the equivalent fluid earth pressure.

Resistance to lateral loads may be developed through friction between the foundation base and the underlying soils and by passive earth pressure along buried foundation components. Friction along the base of the foundation may be computed using a coefficient of friction of 0.5 applied to vertical dead-load forces. This value is based on an effective friction angle of 26 degrees between the soil and the base of the dam. An appropriate factor of safety should be applied to this value. Higher friction coefficients are possible depending on the foundation preparation at the dam/subgrade interface.

In addition to base friction, passive pressure along the downstream dam wall or cutoff may also be used to resist lateral loads. An equivalent fluid earth pressure of 250 pcf times the height of soil is considered appropriate for the passive case under submerged conditions. The depth of soil in the passive zone should be reduced to reflect the potential for scour in this zone. The actual reduction will depend on hydraulic conditions and the scour resistance of the soil.

Values presented above for the active and passive earth pressures do not include a factor of safety. We recommend that the structural engineer include an adequate factor of safety in the design of structures which need to resist lateral loads.

SEEPAGE

GeoEngineers completed seepage analyses below and around the existing diversion dam to develop baseline seepage rate estimates for use by the design team during the preliminary design phase of the project. These analyses are based on the existing dam configuration. Seepage rates will be effected by the replacement dam configuration. Therefore, additional seepage analyses are recommended during the final design phase of the project when details of the replacement diversion dam design are finalized. We also completed preliminary seepage analyses downstream of the proposed temporary cofferdams to be used to divert water during construction. A discussion of these analyses is presented in the "CONSTRUCTION CONSIDERATIONS" section of this report.

The permeability of the native sandy gravel located at the elevation of the base of the existing diversion dam was estimated based on the results of slug (permeability) tests performed in the well installed within boring B-3 and also empirical relationships. The well screen in B-3 extends from the elevation of the base of the dam to approximately ten feet below the base. The fines content of the soil samples recovered from within this zone in boring B-3 ranged from 3 to 7 percent. Permeability, measured in the slug tests, ranged from about 1.0×10^{-2} to 3.4×10^{-2} cm/sec (centimeters per second). The permeability of the native materials was also estimated using Hazen's Formula, an empirical relationship correlating permeability with soil grain size. This approach indicates a range of permeabilities from 1.0×10^{-2} to 1.0×10^{-1} cm/sec for the soils encountered.

Soil grain-size tests performed on representative samples obtained from borings drilled through the dam indicate that the percent fines content ranges from about 2 to 7 percent for the near-surface sands and gravels (3 to 9 feet below the base of the dam) and from about 4 to 17 percent for the underlying mudflow sediments. Isolated pockets of cleaner material may also be present within the Osceola Mudflow deposits as evidenced by the artesian conditions encountered in boring B-1. For the seepage analysis we have assumed that the average permeability of the foundation soils ranges from 1×10^{-2} cm/sec to 1×10^{-1} cm/sec.

We used a standard flow net analysis method to estimate the seepage below the existing dam. The flow net diagram is presented in Figure 4. Based on the estimated range of average soil permeability, we calculate the range in seepage under the existing diversion dam to be 1.0 to 10.0 cfs (cubic feet per second). This corresponds to 0.05 to 0.5 percent of the maximum intake (2,000 cfs) into the flume. Using the average permeability obtained from the slug tests (2×10^{-2} cm/sec), a seepage rate of approximately 2.0 cfs under and around the dam is calculated, which corresponds to 0.1 percent of the maximum flume intake. These calculations assume 11 feet of differential head across the dam. Since seepage rate is directly proportional to differential head, seepage rates at other heads may be estimated through linear extrapolation of these values.

Uplift pressures on the base of the existing diversion dam were also estimated using a flow net analysis. Our results are shown in Figure 4. The largest uplift pressures along the base of the dam occur immediately downstream of the upstream cutoff wall. Assuming 11 feet of differential head we estimate the maximum uplift pressure at this location to be on the order of 600 psf. The uplift pressures decline linearly along the base of the dam to an estimated value of 250 psf just upstream of the downstream cutoff wall. These estimates assume that there is no alleviation of uplift pressure along the base of the dam foundation. Drawings of the existing foundation, however, indicate two means of pressure alleviation below the dam. The first consists of 2-foot-square holes at 6-foot on-center in the concrete base. The second consists of

10-inch-square openings at 6-foot on-center in the downstream concrete cutoff wall. While these openings may reduce water pressure, we assumed conservatively that their effect would be minimal on the overall uplift pressure below the dam.

Although it is difficult to document at this time due to the lack of visual observations, it is our opinion that the potential for piping below or around the dam is relatively low based on the small differential head across the dam and the grain-size distribution of the native soils. The loss of soil below the downstream toe of the dam is most likely caused by water turbulence eroding the soil in this area. Evidence of piping below the existing dam structure should be evaluated during final design.

We understand that sheet pile cutoff walls will be used to reduce piping along the downstream side of the replacement dam. Cutoff walls can be effective in reducing seepage and seepage forces under the dam by lengthening the flow path below the dam. For the replacement dam, which will be a similar width as the existing dam, the proposed cutoff wall depth of 12 feet is appropriate, in our opinion. The contractor may encounter difficulty driving sheet piling to this depth if large cobbles or boulders are encountered. This is discussed in more detail in a subsequent section entitled "CONSTRUCTION CONSIDERATIONS."

Our estimates of seepage quantities, uplift pressures, and piping potential are based on the geometry and hydraulic conditions for the existing diversion dam. We anticipate that seepage, uplift and piping will be reduced by construction of the sheet pile cutoffs for the replacement dam. We recommend that a scope of services during the final design phase include the evaluation of seepage, uplift and piping for the final dam configuration.

NORTH DIKE CONSTRUCTION

The existing north dike alignment begins at the right dam abutment and extends upstream along the north bank of the river a distance of about 2,500 feet (refer to Figure 1). The proposed north dike alignment will also begin at the right dam abutment, but will extend upstream along the existing access road a distance of about 800 feet. At that point, the proposed dike alignment will follow the existing dike alignment. Proposed construction of the north dike for will including raising elevation by 2 to 5 feet along the 800-foot-long access road and along approximately 500 feet of the existing north dike (from about Station 10+00 to 15+00 along the existing alignment, Figure 1). The proposed dike addition will overlie the existing access road between Station 0+00 and 5+00 of the proposed alignment. Because of space constraints, the new fill for the proposed dike addition will be placed primarily over native soils south of the existing access road fill between Station 5+00 and 8+00.

Work on the east 1,000 feet (Station 15+00 to 25+00 along the existing alignment) of the existing dike will be accomplished under a separate contract relative to construction of the proposed water intake structure for the fish hatchery. Recommendations in this section of the report are limited to north dike construction from Station 0+00 to 13+00 along the proposed dike alignment. References to stationing in this section pertain only to the proposed alignment stationing shown in Figure 1.

The existing north dike (from Station 8+00 to 13+00) is typically constructed of gravelly sands which are loose to medium dense and overlay existing alluvial deposits of sand and gravel. We estimate the side slopes of the dike to be currently at between 2:1 and 3:1 (horizontal to vertical). Typical cross-sections are presented in Figure 5. Much of the dike is lined with riprap or concrete.

No explorations have been completed in or near the access road. Based on evaluation of surface soils, we anticipate that the road is comprised primarily of sand and gravel with relatively minor percentages of silt. The river side of the 2- to 4-foot-high road is protected by 12-inch - minus quarry spalls.

It is our opinion that the dike itself and the underlying native soils are inherently stable at the existing configuration. The existing access road also appears to be inherently stable and suitable for placement of additional fill. All trees and other vegetation in the existing dike and within the zone of proposed fill placement should be removed to ground level. We recommend that the near-surface soils of the existing dike and access road be well compacted using a vibratory roller prior to placing any fill. Some areas along the toe of the access road or dike may be underlain by soft, native soil deposits. Any soft or loose areas should be removed and replaced with compacted structural fill. A determination of the suitability of subgrade soils should be made at the time of construction.

The height of the existing dike and access road may be increased by placing suitable fill over adequately prepared subgrade. Widening of the dike will be required in some areas where the top of the existing dike is narrow. Specific design recommendations regarding fill thickness, material specifications and riprap protection will depend on the location along the dike or access road, new dike elevations and the existing condition of the dike and access road. We present conceptual recommendations for increasing dike height in Figure 5.

We recommend that the fill consist of sand and gravel containing less than about 12 percent fines (percent passing the number 200 sieve, by dry weight). The fill should be compacted in lifts not exceeding 12 inches loose thickness to a minimum density of 90 percent of the maximum dry density as per ASTM D-1557. We anticipate the fill may be obtained from various on-site and off-site sources. We understand that construction scheduling may be such that material for the earth cofferdam for Phase I (discussed in a subsequent section entitled "CONSTRUCTION CONSIDERATIONS") could be used for the north dike construction. Also, dredge soil from the nearby Walsh Basin pit is available. Based on our preliminary evaluation of these materials, it is our opinion that they would be suitable for north dike construction. A geotextile fabric or a suitable thickness of a soil filter blanket may be needed along the river side slopes to prevent finer fill material from washing out through the riprap.

The south slope of all new fill areas for the north dike should be protected with riprap. Based on an evaluation of the performance of existing riprap in this area of the river, we recommend that the riprap consist of sound, angular rock with a maximum size of 24 inches and a mean size of 12 inches. The existing riprap along the south slope of the access road is considered suitable for permanent riprap protection of the new dike. We recommend that this riprap be removed prior to dike construction and stored on site for later use along the dike slopes. Elsewhere, riprap will need to be imported from a suitable source which should be approved by GeoEngineers prior to transport of riprap to the site.

Below the new fill, we recommend that riprap be used only to fill in relatively large voids in between the larger rock or where riprap is absent in the older existing dike sections. In areas where existing riprap appears to be stable new riprap will probably not be required. After one to two winter seasons, the performance of the dike should be evaluated and repairs made as necessary.

RIVER CHANNEL AND DIKE EROSION PROTECTION

The riverbanks and the north dike located within a half mile upstream of the dam appear to be fairly stable at this time. Some minor erosion and bank recession appear to be present along the south bluff just upstream of the dam. Erosion of this bluff has occurred relatively slowly historically based on the review of the aerial photographs. It is not possible to accurately estimate changes in historical erosion rates from air photographs due to lack of clarity. The average rate of bank erosion appears to have been about 0.1 to 0.2 feet per year over the last 50 years. It is likely that bank erosion will continue at this rate, provided there are no changes in river flow or geometry. We anticipate that bank erosion will be intermittent with slabs of soil falling into the river during high flow rates and little to no erosion at other times. The rate of bank erosion may increase if upstream river conditions (including flow rates or river geometry) change.

It is possible that a slight increase in bank erosion may occur over historical rates due to a narrower channel and slightly sharper bend that has developed in this part of the river over the last 20 years. However, we do not expect any rate increase to be significant. Based on our findings, we do not anticipate that bank erosion protection will be needed along the south bluff if historical erosion rates, or slightly higher, are acceptable. Should the rate of bank erosion increase in the future, the need for bank erosion protection should be evaluated.

We recommend that a monitoring system be installed at the top of the bluff in this area to obtain more information on the erosion rate. The monitoring system may consist of steel stakes driven into the ground and surveyed, horizontally and vertically. The monitoring frequency should be at least annually for the first few years and, depending on the results, may be reduced subsequently.

If the historical erosion rates are judged to be unacceptable and additional riverbank protection is deemed appropriate, it may be accomplished by several methods. These include flattening the bluff and protecting the slope with suitable riprap material or constructing training groins. Slope flattening may be accomplished by either placing fill along the toe or by cutting back from the top of the bluff. Training groins, if used, will require placement of large rock (i.e., 2 to 3 feet in size) in the river in a configuration such that sediment will naturally be deposited behind the groins. We anticipate that the upstream tie-in point to the riverbank will be approximately 1,000 to 1,200 feet upstream of the diversion dam. Details of the groin configuration may be developed as needed.

The south bank upstream of the bluff is covered with well-established vegetation. Erosion protection of the south bank beyond about 1,000 feet upstream of the dam does not appear to be warranted.

Based on our observations and discussions with King County and Puget personnel, we consider the level of slope protection along the north bank 1,800 to 2,500 feet upstream of the dam to be marginally adequate. The existing riprap (about 2-foot maximum size) appears to be protecting the slope satisfactorily from excessive erosion. However, since the dike was constructed in 1968, repair and replacement of portions of the rock protection has been required on at least four occasions. Apparently only moderate losses of protection have occurred after any one high flow period and the overall integrity of the dike has remained intact.

We consider that two options for protecting this portion of the existing dike are available. The first is to continue with the existing dike protection program with the understanding that repair and replacement of portions of the protection will be required every few years. The second option involves overlaying existing dike surfaces with a properly designed erosion protection system. In areas where riprap is absent or of insufficient thickness, we envision the system may involve two layers of material. A layer of large diameter (about 3 feet) riprap should overly a layer of smaller diameter well-graded crushed rock. This smaller rock would serve as filter material to reduce erosion/ piping losses of material through the larger rock. Where the existing riprap is present in sufficient thickness, we anticipate that the large diameter riprap can be placed directly over the existing riprap. Use of larger riprap may be expensive since it is our understanding from King County Flood Control representatives that larger riprap is not available locally. Continued use of existing smaller riprap with periodic maintenance may be more cost-effective.

The older portion of the north dike that will still be used and is located from 1,000 to 1,300 feet upstream of the dam appears to be stable with little to no evidence of erosion. Vegetation is well-established on the dike slopes and most of the shoreline in this area is somewhat protected by the sand and gravel bars on this side of the river. Therefore, it is our opinion that additional erosion protection is not warranted in the older portion of the north dike except in areas where significant voids exist between large rocks or where riprap is absent (refer to previous section entitled "NORTH DIKE CONSTRUCTION").

CONSTRUCTION CONSIDERATIONS

This section addresses geotechnical aspects of the construction of the proposed replacement dam. Our understanding of the construction schedule, methodology and sequencing as it is presently envisioned, is based on discussions with Mr. Bob King and on information in Draft Technical Memorandum No. 12 by HDR Engineering, dated December 24, 1991.

We understand that the project will be constructed in two phases over a one-year period. Construction in the White River will require the use of temporary cofferdams upstream of the site to divert the river around the project area. The cofferdams will be designed for a water elevation of 671 feet with 3 feet of freeboard. The design elevation of the top of the cofferdams will therefore be 674 feet. Excavation depths behind (downstream of) the cofferdams is expected to vary from Elevation 660 feet in the Phase I area to as deep as Elevation 647 feet in the Phase II area. In the following subsections we present a discussion of construction considerations for cofferdams and dewatering.

COFFERDAMS AND TEMPORARY DIVERSION CHANNEL

Phase I

The cofferdam for Phase I will extend from the right abutment of the existing dam approximately 250 feet south. The existing riverbed elevation in this area varies from 671 to 665 feet, corresponding to a cofferdam height of 3 to 9 feet above the present riverbed level. The northern 200 feet of the cofferdam will be in a secondary channel of the river where the water velocity is relatively low. The southern end of the cofferdam will extend approximately 40 to 50 feet into the main channel of the river where the water velocity is high.

The northern 200 feet of the Phase I cofferdam will be an earth dike comprised of sand and gravel. Riprap protection will probably not be needed because of the low water velocity in this area of the river. Heavy plastic sheeting should be placed over the upstream face of the earth cofferdam to reduce water flow into the excavation.

We recommend that the fill material for this portion of the cofferdam consist primarily of gravel with less than 50 percent sand and less than 5 percent fines by weight. Less coarse material will be difficult to place below water. The material in the sand and gravel bars near the north bank of the river and in the bars in the middle of the river is expected to be suitable for this use, provided the necessary permits for dredging can be obtained. The surficial 1 to 2 feet of soil in the ponded bars near the right abutment probably *contain* too much sand and gravel to permit placement in water. We recommend that these siltier soils not be used in cofferdam construction. Any proposed import fill should be evaluated prior to its transport to the site. We anticipate that an earth cofferdam could be constructed with slopes of about 2½H: IV. The actual slope angle that can be achieved will depend primarily on the gradation of the fill material. We anticipate that adequate compaction can be achieved by the action of dozer equipment passing over the fill, provided the material conforms to our recommendations.

The southern 50 feet of the Phase I cofferdam may be constructed using either sheet pile cells or crushed rock fill protected by riprap. It is our opinion that the crushed rock option may be more cost-effective than the sheet pile cells because of the higher costs of driving sheet piles in this type of soil, particularly for such a small area. A discussion of sheet pile cellular cofferdams is presented in the following subsection for Phase II.

The crushed rock, if used, should consist of well-graded, 4-inch-minus angular material with less than 5 percent fines by weight. The purpose of using crushed rock instead of sand and gravel is that the more angular material will allow easier placement and compaction in the higher water velocities. This material will need to be imported from a suitable source. Because of the high water velocity in this area this portion of the Phase I cofferdam will need to be protected from erosion by placing adequately sized riprap along the upstream face. It will likely be necessary to place the riprap in front (upstream) of the cofferdam site prior to placing any crushed rock to provide a partial diversion of the water flow. Temporary lowering of the pool behind the dam (i.e., lowering the river level) would facilitate material placement and is recommended. Once the crushed rock is in place the riprap would be moved over onto the upstream face of the dike.

We anticipate that the riprap will need to be relatively large, ranging from 1 foot up to 3 to 4 feet in size. It will probably not be possible to place plastic sheeting or geotextiles along the upstream face of this part of the cofferdam due to the high water velocities. It may be necessary to drive sheet piles through the crushed rock after the cofferdam is constructed to reduce water flow into the excavation. Alternatively, it may be feasible to install additional wells in this area of the cofferdam. This should be evaluated further during the design phase after the recommended field programs, discussed below, are completed.

A temporary sheet pile cofferdam and diversion channel will be constructed upstream of the existing intake channel as part of Phase I construction. The primary geotechnical concerns related to this construction is the problem that may arise during installation of the sheet piling due to the presence of large cobbles and boulders in the soil. A discussion of sheet pile installation and methods to reduce impacts of cobbles and boulders on pile driving is presented in the following section pertaining to Phase II construction.

Phase II

The cofferdam for Phase I construction will be located entirely in the main channel of the White River. An earth cofferdam is not feasible in this area. The Phase II cofferdam will therefore consist of connecting sheet pile cells filled with sand and gravel. The present riverbed elevation in this area is not accurately known. Based on interpolation between topographic lines we anticipate that the riverbed elevation ranges from about 658 to 664 feet. The planned elevation of the base of the excavation behind the cofferdam ranges from 658 to 647 feet corresponding to depths below existing river bottom of about 2 to 11 feet.

The sheet piles will need to be driven sufficiently deep to provide adequate stability against sliding and overturning and to reduce the potential for scour around the base of the cells. Driving the sheet piles to the required elevations is expected to be very difficult and may be impossible using standard driving techniques due to the presence of cobbles and boulders in a relatively dense matrix. The size of boulders encountered in our explorations and observed in the riverbed ranges from 1 to 4 feet in diameter. The results from NHC's physical model tests indicate that scour depths may be on the order of 8 to 10 feet. Based on these results, we recommend that the sheet pile cells should extend a minimum of about 10 to 12 feet below the present river bottom to provide adequate resistance against scour. Driving to this depth will, as a minimum, require the use of pile tips. In addition, predrilling or pre-spudding may be required where large-sized boulders are encountered. It may also be necessary to clam out material inside the cofferdam as the piles are being driven in order to drive the sheet piles.

The fill material for the cellular cofferdams should consist of sand and gravel with less than 5 percent fines by weight. The native riverbed deposits in this area of the river are expected to be suitable for this use. Import fill, if required, should be similar to the native sand and gravel in the river. A preliminary value for the coefficient of friction of 34 degrees and a wet unit weight of 130 pcf can be used for conceptual design of the cells. Refinement of these values should be made after the actual fill material has been selected and evaluated.

DEWATERING

Temporary dewatering will be required to control ground water seepage into the excavations. During Phase I construction we anticipate that 2,000 to 6,000 gpm may need to be removed from the excavation, assuming that the southern end of the cofferdam will be constructed using crushed rock without any cutoff. These flow rates are based on the assumption that soils underlying the cofferdam are relatively permeable to depths on the order of 40 to 50 feet. Lower flow rates are expected if the mudflow deposits anticipated below about 9 feet in depth are less permeable than assumed. In addition, significantly less water seepage (about one half) would be expected if the southern portion of this cofferdam is constructed using sheet pile cells or if a sheet pile cutoff is installed through the crushed rock fill. Because of the plastic sheeting along the northern portion of the earth cofferdam most of the seepage into the excavation will originate from soils below the cofferdam.

During Phase II construction, seepage will be primarily below the cellular cofferdams. If these cofferdams extend sufficiently deep into the low permeability mudflow deposits they will act as a partial cutoff and seepage flow will be reduced. If higher permeability soils are present around and below the cells or the installation of the sheet piles results in zones of coarse material around the piles, then higher seepage will be expected. Based on preliminary flow net analyses we estimate that total seepage into the Phase II excavation may range from 4,000 to 8,000 gpm.

Because of the relatively high seepage rates we recommend that dewatering be accomplished using either wells or well points. At this time we anticipate that the well tips will need to extend to a depth of about 40 to 50 feet below the base of the cofferdams. In Phase I, the wells should be installed along the downstream side of the cofferdam. In Phase II the wells should be installed through the middle of the cells. Shallow sumps with pumps may also be required at the base of the excavation to intercept local seepage. Artesian conditions were encountered in one boring (B-2) at an elevation of about 631 feet (30 feet in depth). While planned excavations are not this deep, it is possible that artesian conditions may be encountered at shallower depths. If artesian ground water is encountered during construction, additional wells may be required inside the excavation. Flow rates could increase by several thousand gallons per minute if significant artesian water is encountered.

The size, spacing and design of the wells will need to be addressed during the final design phase. GeoEngineers will be available to provide additional geotechnical input to dewatering design after more information is obtained during the final design phase. This is discussed in more detail in a subsequent section entitled "RECOMMENDED FUTURE GEOTECHNICAL SERVICES."

RECOMMENDED FUTURE GEOTECHNICAL SERVICES

The conclusions and recommendations presented above are based on widely spaced exploration data and preliminary design drawings. The in-river project construction period will likely be limited to a few months based on regulations governing fisheries and the flow conditions in the river itself. This will result in a very tight project construction schedule. With this in mind, it seems prudent to carry out appropriate test programs during final design to confirm geotechnical conditions in order to reduce the potential for negative construction impacts and delays. We have accordingly identified several test programs to meet this objective and also identified additional geotechnical engineering services which will be required during the final phase of design.

FIELD AND LABORATORY TEST PROGRAMS

1. Test pile program - A test pile program should be implemented to evaluate the feasibility of driving sheet piles in the riverbed conditions, both upstream and downstream of the dam and in the area of the proposed temporary diversion. This would allow evaluating various construction techniques such as predrilling, spudding and using pile tips to drive sheet piles into the gravel, cobble and boulder layer. From this program we would be able to develop more specific recommendations for construction of the cellular cofferdams.
2. Test seepage program - A small excavation should be made in the sediment adjacent to the flashboards at the right abutment to monitor seepage through the sediment and native riverbed soil below the bottom of the excavation. The excavation may be readily dewatered by removing the flashboards. Seepage will be observed as the excavation is dewatered.
3. Pump test program - One or more pump tests should be conducted in either an existing well or in a new well installed in the test excavation area (or both). The pump test would allow evaluation of site-specific seepage rates, drawdown characteristics, effectiveness of the well design and pumping requirements for use in the design of a temporary dewatering system during construction (discussed further below).

4. Concrete coring program - About two-thirds of the replacement dam will utilize the existing dam concrete slab foundation and underlying soils for support. Based on the two borings drilled through the existing dam we do not anticipate the presence of voids beneath the concrete slab; however, because of the potential variability of conditions at this site and because of the potential adverse impact on the construction schedule of the replacement dam if voids are present, we propose to core about six to eight small-diameter holes through the existing dam apron. This will allow evaluation of the thickness of the concrete, quality of the concrete, and the presence or absence of voids (due to piping erosion) below the slab.
5. Additional boring program - No subsurface information is available for the proposed control building site located upstream of the dam. In addition, more information is needed regarding the nature of the soils along the proposed downstream sheet pile cutoff. In particular, it will be important to obtain more information regarding the depth to and the amount of silt in the mudflow deposits to properly evaluate potential scour in this area of the dam. We therefore recommend that additional borings be drilled for the project. One or two borings may be appropriate at the proposed control building site. One boring every 50 to 60 feet along the downstream side of the existing dam is also recommended.
6. Additional onshore test pit program - The proposed alignment for a portion of the north dike has been moved north along the existing access road since our first test pit program. There are no test pits near or in the existing access road. We therefore propose to excavate four to eight additional test pits along the toe of and in the access road to evaluate soil conditions with regard to ability in proposed fill areas. The test pits would be accomplished using the excavator designated for the test seepage program discussed above.
7. Geotechnical laboratory test program - The use of material from on-site (riverbed deposits) and near-site (Walsh basin dredge material) is planned for the earth cofferdam, inside the cellular sheet pile cofferdam and for the north dike construction. We recommend that representative samples be obtained from these and other potential borrow sources, if applicable, for laboratory testing. Tests would include moisture content determinations, grain-size analyses and maximum dry density determination.

ENGINEERING SERVICES

1. Final design recommendations - Once more details regarding the replacement dam components are available we will review preliminary recommendations presented herein and refine or revise them as appropriate.
2. Settlement analyses - We understand that the radial gates are very sensitive to post-construction settlements. It is our opinion at this time that, with proper preparation, the in-situ soils at the site will likely provide adequate support for these structures. Potential settlement of the radial gates should be evaluated once final design dead and live loads are available to see that the estimates are acceptable.

3. Refined seepage analyses - The seepage analyses presented herein are based on the existing dam foundation. Estimate flow rates below the replacement dam should be made when the final details of the replacement dam are available.

4. Dewatering design - We understand that construction scheduling will be very important to the success of the project. Regulations governing construction in the river may limit the construction schedule. Therefore, it will be important that the contractor install a dewatering system that has surplus capacity to remove water from the excavation. Delays due to inadequate dewatering may adversely impact the construction schedule. Typically, the contractor is made responsible for the design as well as construction of the dewatering system. The costs for this work are typically bid on a lump sum basis. Because of this, it is common for the contractor to design a dewatering system with a low or no factor of safety. If the system is inadequate, then additional wells are added later. This can result in construction delays. To reduce potential construction delays, we recommend that the design of the system be a coordinated effort between the design team and the contractor. Because of the potential unknowns, we also recommend that the contract be on a time-and materials basis to increase the contractor's incentive to develop an adequate system the first time. Data from the pump tests (described above), together with input from the contractor, should be used to develop recommendations for the design and construction of the appropriate dewatering system. This should reduce potential change orders during construction.

5. Control building - Temporary and permanent cuts will be required along the south bluff for the proposed control building. Recommendations regarding cut slope stability, minimum slope angles, temporary retaining structures and permanent walls will be required. Once details regarding the location, depth and space requirements of the excavations in this area are known we can develop design geotechnical recommendations. Specific recommendations regarding additional explorations will be made during the final design phase.

LIMITATIONS

We have prepared this report for use by Puget Sound Power & Light Company and HDR Engineering Inc. in design of a portion of this project. The data and report are based on preliminary reconnaissance and design information and our conclusions and interpretations should not be construed as a warranty of the subsurface conditions. We will prepare a scope of services together with estimated costs for the final design phase in a separate proposal.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No other conditions, express or implied, should be understood.

We trust this information meets your needs. If you have any questions regarding this information, please contact us.

Respectfully submitted,

GeoEngineers, Inc.

5/4/92

EXPIRES 3/11/93

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DWM:GMD:wd
Document ID: **0186115.pr**

16 copies submitted

LIST OF REFERENCE REPORTS
WHITE RIVER PROJECT

GeoEngineers, Inc. "Preliminary Report - Headwater Benefits, White River Basin," June 3, 1983.

GeoEngineers, Inc. "Report - Reconnaissance for White River Flume," July 16, 1984.

GeoEngineers, Inc. "Report - Preliminary Osceola Mudflow Exploration for Proposed Powerhouse at the White River Lined Canal," March 21, 1985.

GeoEngineers, Inc. "Report of Geotechnical Consultation, Preliminary Embankment Stability Basins, White River Sediment Basins," June 6, 1985

GeoEngineers, Inc. "Report - Geotechnical Investigation, Proposed Transformer Addition, White River Plant," February 14, 1986.

GeoEngineers, Inc. "Progress Report No. 1 - Design Memorandum, Geotechnical Services, Trailrace Elements, White River Power Project," June 24, 1986.

GeoEngineers, Inc. "Report of Geotechnical Services, Subsurface Contamination Study, White River Headworks," February 27, 1987.

GeoEngineers, Inc. "Report, Geotechnical Studies, Phases I & II, White River Flume Rebuild, Headworks Section," August 31, 1987.

Sverdrup Corporation "Draft Report - Detailed Hatchery Siting Study On or Near the White River," June 1987.

Ott Water Engineers, Inc. "White River - Facility Siting Study," August 1986.

Ebasco Services, Inc. "Lined Canal Replacement for White River Project," January 1989.

Puget Power "Application for License, Major Project at Existing Dam, white River Project," November 1983.

Dunne, Thomas "Sediment Transport and Sedimentation Between RM's 5 and 30 Along the White River, Washington," August 1986.

Ebasco Services Inc. "White River Project Sediment Studies," July 1988. Ebasco

Services Inc. "Seismic Refraction Survey," November 1982.

Hydrocomp Inc. "Analysis of the Effects of Probable Maximum Precipitation on the White River Project," August 1983.

Golder Associates "Geotechnical Engineering Investigation, White River Hatchery, Buckley, Washington," August 1988.

TABLE 1
SUMMARY OF TREES ALONG EXISTING NORTH DIKE ALIGNMENT

Disturbance From Abutment (feet)	Number of Trees Within Existing Dike					Number of Trees Within 15 feet of North Toe				
	4"-8"	8"-12"	12"-18"	18"-36"	36" +	4"-8"	8"-12"	12"-18"	18"-36"	36" +
0-100										
100-200	10		3			2	0	1	1	
200-300	3	1	4	3		4				
300-400	4	2	9	3	1					3
400-500	8	4	5	3				2		
500-600			2	2	2				2	1
600-700		2	3	1			8	1	1	
700-800		3	6	1	3	1	3	1		1
800-900	2	1								
900-1000	1									
1000-1100	4	6	2					1		
1100-1200	3	11	4	3	1	2			1	
1200-1300	8	12	2	1		4	3			
1300-1400	7	10	5	4	1	2	1		1	
1400-1500	11	3	4		1	12	1		1	
1500-1600		3		1		6	1	2		3
1600-1700	5					5	1	4	3	3
1700-1800						2	5	1	1	4
1800-1900						10	5			

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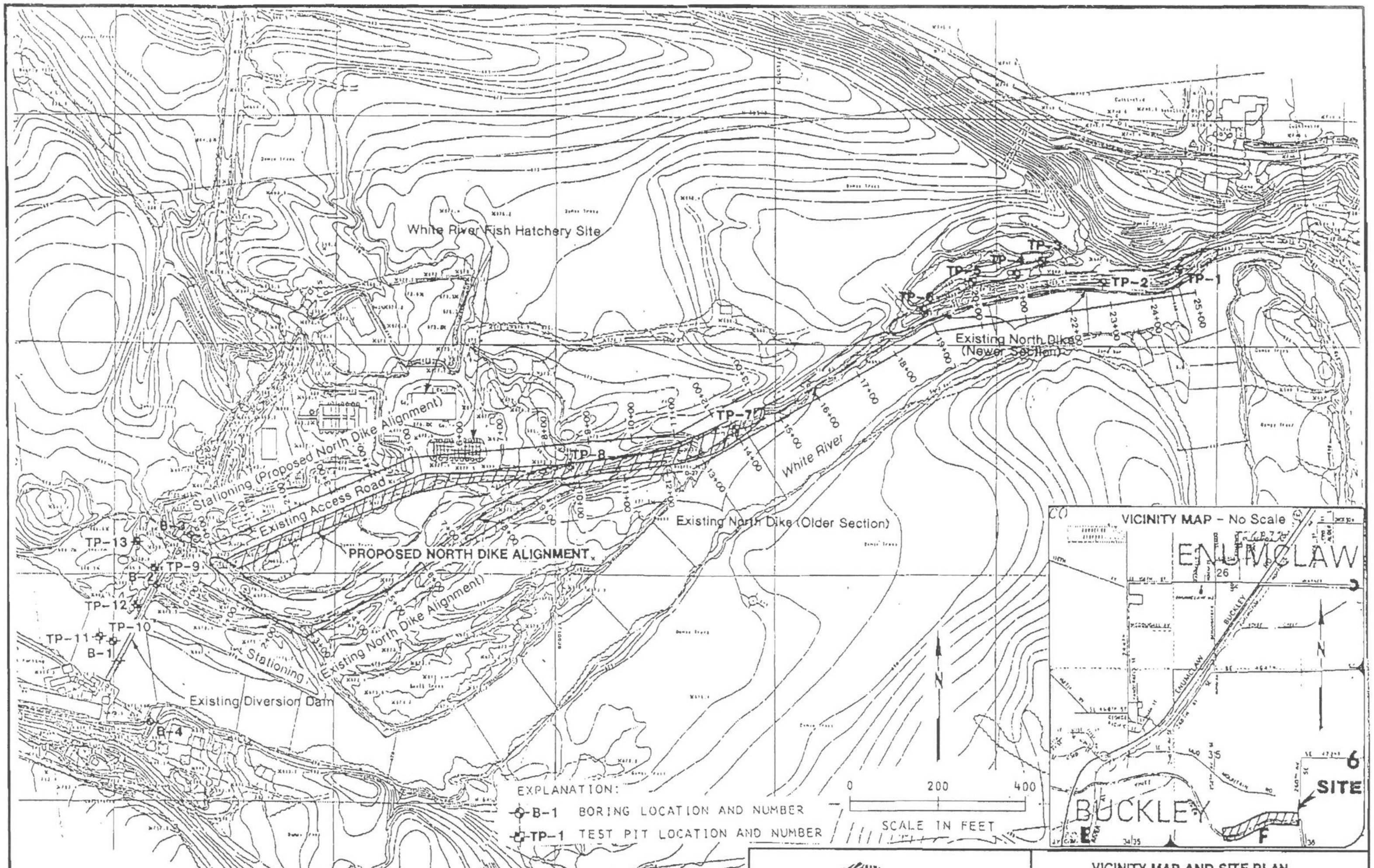

REFERENCE: PHOTOGRAMMETRIC MAP ENTITLED "WHITE RIVER", DATED 12/11/89, BY WALKER AND ASSOCIATES, INC.

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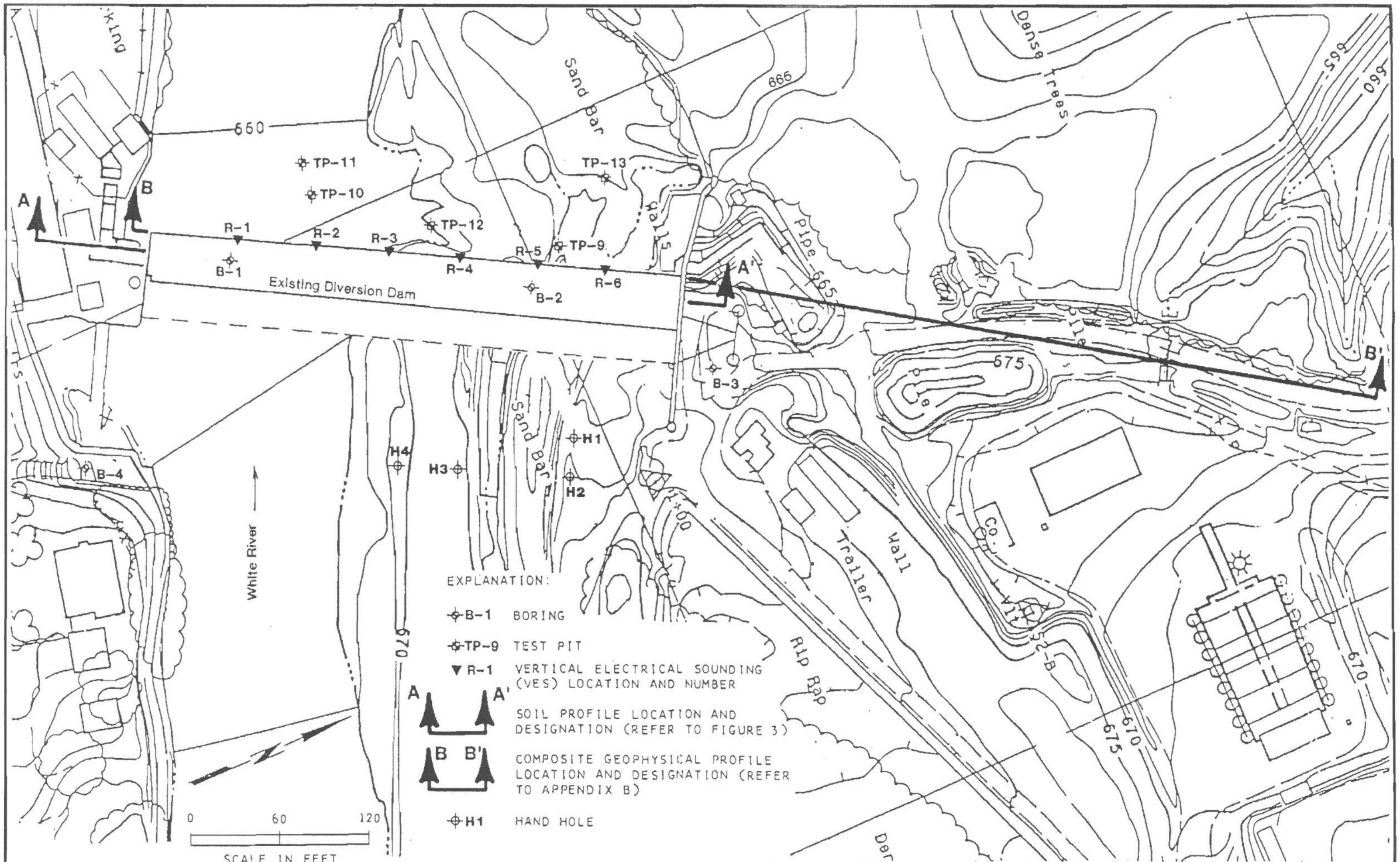
VICINITY MAP AND SITE PLAN

FIGURE 1



REFERENCE: PHOTOGRAMMETRIC MAP ENTITLED "WHITE RIVER", DATED 12/11/89, BY WALKER AND ASSOCIATES, INC.

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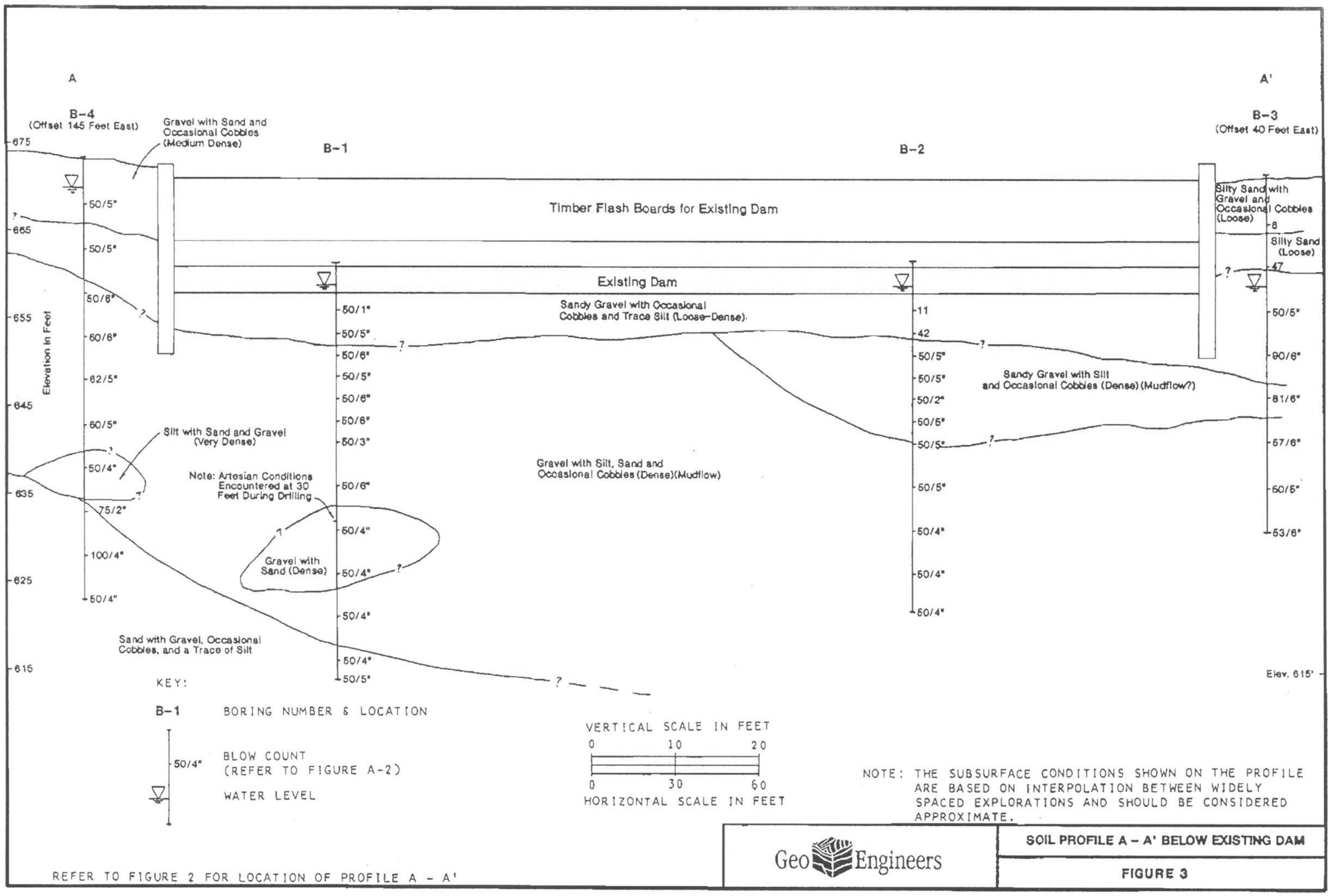
SITE PLAN
DIVERSION DAM SITE

FIGURE 2

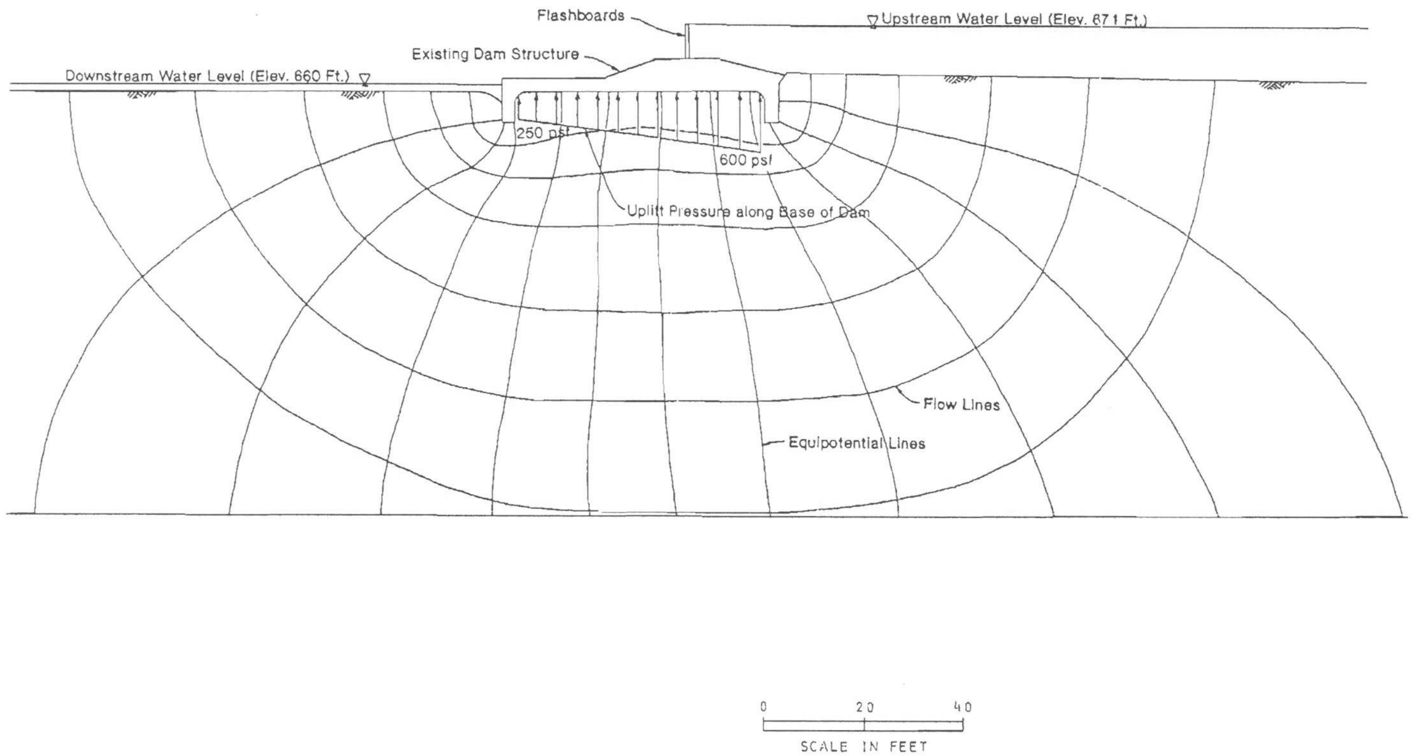
0186-115-B06 DJC:KKT 9-24-90

0186-115-B06 DJC:BOH 10.17.90



SOIL PROFILE A – A' BELOW EXISTING DAM

FIGURE 3



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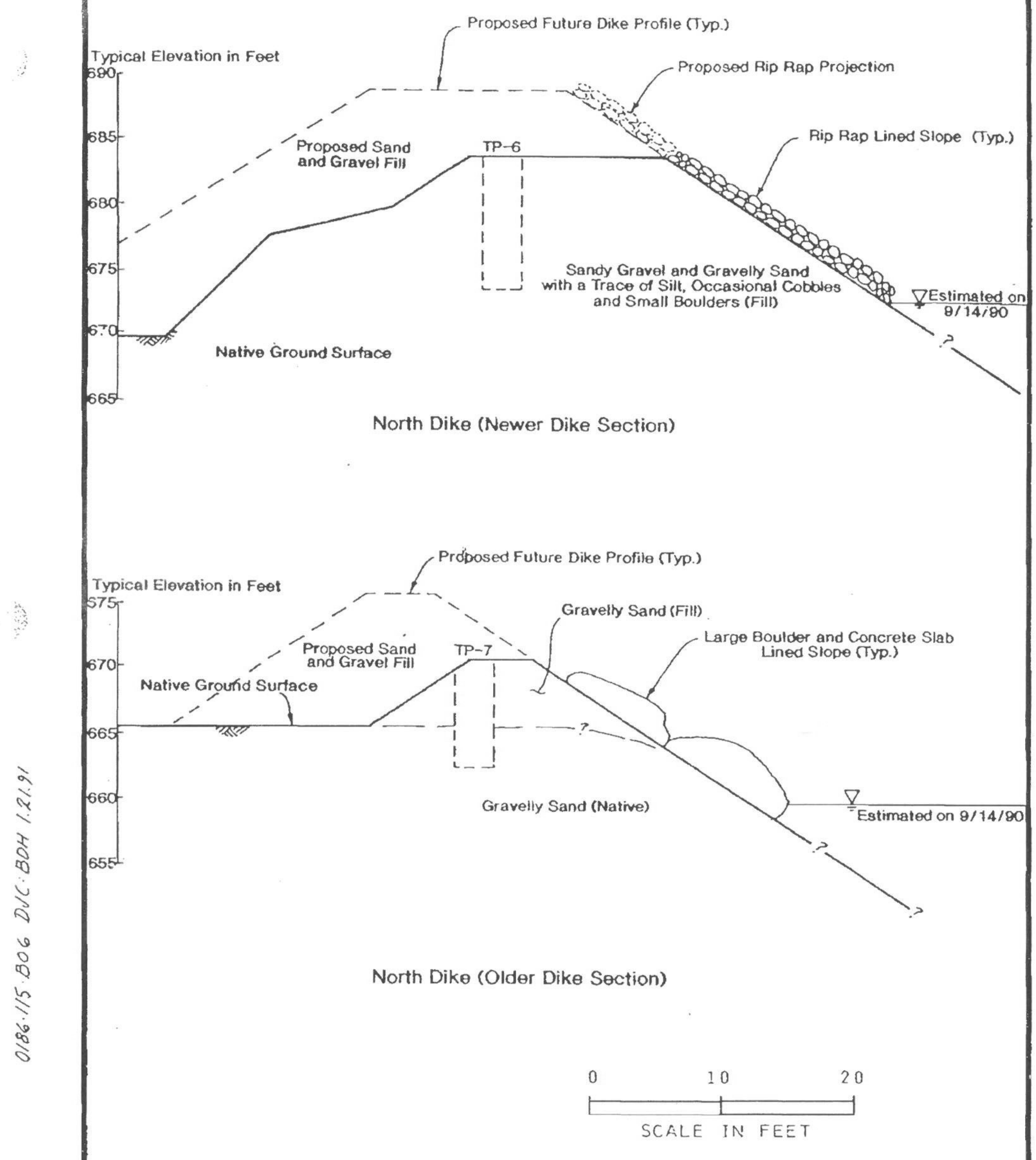
FLOW NET FOR EXISTING DAM

FIGURE 4

0186-115-B06 DWM BDH 1.22.91



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TYPICAL NORTH DIKE CROSS SECTIONS

FIGURE 5

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Subsurface explorations at the site were explored by drilling four borings from August 28 to September 4, 1990, excavating 8 test pits on September 14, 1990, excavating five test pits on August 2, 1991 and digging four hand holes on April 16, 1992. The four borings were drilled to depths ranging from 40 to 50 feet using a truck-mounted air-rotary drill rig. Relatively undisturbed samples were obtained from the borings using a 3.25 inch O.D. split-barrel sampler driven into the soil using a 300-pound hammer falling a distance of approximately 30 inches. The number of blows required to drive the sampler the last 12 inches, or other indicated distances, is recorded on the boring logs. The locations of the borings are shown on the Site Plans, Figures 1 and 2.

Test pits TP-1 through TP-8 were excavated in the east portion dike along the north bank of the river with a rubber-tired backhoe to depths ranging from 7 to 10.5 feet. Test pits TP-9 through TP-13 were excavated in the river bed downstream of the existing dam using a track-mounted excavator. The depths of these five test pits ranged from 7 to 16 feet below the river bed. The locations of the test pits are shown on the Site Plan in Figure 1.

Hand holes H-1 through H-4 were excavated in sand bars in the river by digging with a hand shovel. The depth of the holes ranged from 1 to 3 feet. Location of the hand holes is shown in Figure 2.

The explorations were either completed or were continuously monitored by a geotechnical engineer from our staff who selected sample intervals, examined and classified samples recovered, and kept a log of each boring based on examination of the samples. Exploration locations were measured by taping and pacing from existing survey markers located in the field. Ground surface elevations at the explorations have been interpreted from contours shown on a photogrammetric map constructed by Walker and Associates, Inc. entitled "White River," dated December 11, 1989.

The soils encountered in our explorations were classified visually in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is presented in Figure A-2. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. They also indicate the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples, it was interpreted. The boring logs are presented in Figures A-3 through A-8. Test pit logs are presented in Figures A-9 through A-14. A description of the soils encountered in the four hand holes is presented in Figures A-15.

Site ground water conditions were observed as the explorations were performed. These observations are presented on the boring and test pit logs. The water level in the well installed within boring B-3 was also recorded.

LABORATORY TESTING

All soil samples were brought to our laboratory for further examination. Selected samples were tested to determine grain size characteristics. Mechanical grain-size analyses were performed on thirteen representative soil samples. Gradation curves for these samples are presented in Figures A-16 through A-22. Nine additional samples were tested for percent fines (material passing the number 200 sieve). The percent fines test results are presented in Figure A-23. The laboratory tests which were performed on the soil samples are also indicated on the boring logs.

SLUG TESTS

A series of slug tests was performed on the 2-inch-diameter well installed within boring B-3. Slug test numbers 1 and 2 were performed by adding a one and four gallon slug of water to the well, respectively. A solid 1.25-inch-diameter rod was used as the slug for test numbers 3 and 4. The rod was inserted and removed from the well for the respective tests. Water level measurements were measured and recorded with a data acquisition system consisting of a pressure transducer, a data logger, and a portable personal computer. The field data was reduced using Hvorslev's method and the Bouwer-Rice method. The results are presented in Figure A-24.

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS MORE THAN 50% PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1. Field classification is based on visual examination of soil in general accordance with ASTM D2488-83.
2. Soil classification using laboratory tests is based on ASTM D2487-83.
3. Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water

Wet - Visible free water or saturated, usually soil is obtained from below water table

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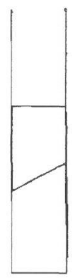
SOIL CLASSIFICATION SYSTEM

FIGURE A-1

LABORATORY TESTS:

- AL Atterberg limits
- CP Compaction
- CS Consolidation
- DS Direct shear
- GS Grain – size
- %F Percent fines
- HA Hydrometer analysis
- SK Permeability
- SM Moisture content
- MD Moisture and density
- SP Swelling pressure
- TX Triaxial compression
- UC Unconfined compression
- CA Chemical analysis

SOIL GRAPH:



SM Soil Group Symbol (See Note 2)

Distinct Contact Between Soil Strata

Gradual or Approximate Location of Change Between Soil Strata

Water Level

Bottom of Boring

BLOW-COUNT/SAMPLE DATA:

Blows required to drive a 2.4-inch I.D. split-barrel sampler 12 inches or other indicated distances using a 300-pound hammer falling 30 inches.

- 22 Location of relatively undisturbed sample
- 12 Location of disturbed sample
- 17 Location of sampling attempt with no recovery

Blows required to drive a 1.5-inch I.D. (SPT) split-barrel sampler 12 inches or other indicated distances using 140-pound hammer falling 30 inches.

- 10 Location of sample obtained in general accordance with Standard Penetration Test (ASTM D-1586) procedures
- 26 Location of SPT sampling attempt with no recovery

Location of grab sample

"P" indicates sampler pushed with weight of hammer or against weight of drill rig.

NOTES:

1. The reader must refer to the discussion in the report text, the Key to Boring Log Symbols and the exploration logs for a proper understanding of subsurface conditions.

2. Soil classification system is summarized in Figure A-1.

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KEY TO BORING LOG SYMBOLS

FIGURE A-2

TEST DATA

BORING B-1

Surface Elevation (ft.): 661.0

Depth (ft)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
0							Rock and concrete filled timber crib dam
~3						GM	Dark gray sandy fine to coarse gravel with a trace of silt and occasional cobbles (dense, wet)
~5				50/1"			
~7.5	GS			50/5"		GP GM	Gray fine to coarse gravel with silt, sand and occasional cobbles (dense, wet)
~10				50/6"			
~12.5				50/5"			Driving on a cobble
~15				50/6"			
~17.5	%F			50/6"			
~19						SP SM	Dark gray gravelly medium to coarse sand with silt (dense, wet)
~20				50/3"			
~25				50/6"			
~28						GW	Dark gray fine to coarse gravel with sand, occasional cobbles and a trace of silt (dense, wet)
~30	%F			50/4"			
~35	GS			50/4"			
~37.5						GP GM	Gray sandy fine to coarse gravel with silt and occasional cobbles (dense, wet)
40							

DEPTH IN FEET

Note: See Figure A-2 for explanation of symbols

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Log of Boring

Figure A-3

8-115-B06 GMD:DJC:CDG 10/16/98

TEST DATA

BORING B-1
(Continued)

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow-Count	Samples	Group Symbol	DESCRIPTION
%F			50/4"	■	SP	Dark gray medium to coarse sand with gravel (dense, wet)
			50/4"	⊠		
			50/5"	⊠		

DEPTH IN FEET: 40, 45, 50, 55, 60, 65, 70, 75, 80

Boring completed at 50.5 feet on 8/29/90
Ground water encountered at approximately 1 foot depth at time of drilling
Artesian water conditions encountered at 30 feet caused the water level in the boring to rise to about 5 feet above the top of boring during drilling

Note: See Figure A-2 for explanation of symbols

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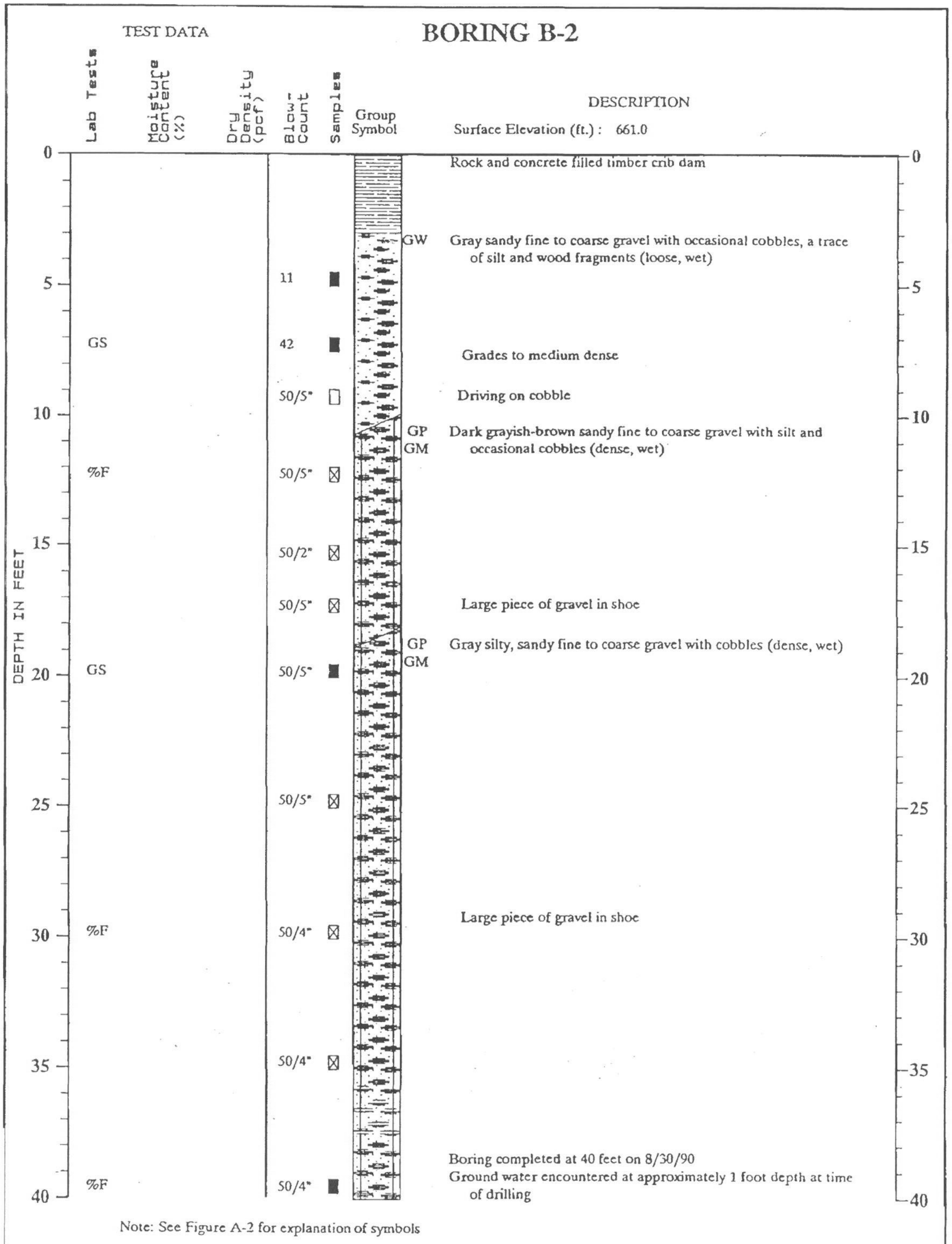
Log of Boring

Figure A-4

86-115-B06 :GMD:DJC:CDG 1/22/91



Note: See Figure A-2 for explanation of symbols

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Log of Boring

Figure A-5

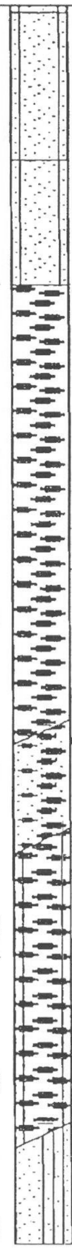
86-115-B06 :GMD:DJC:CDG 10/18/90

BORING B-3

TEST DATA

Surface Elevation (ft.): 673.0

Depth (ft)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
0						SM	Dark brown silty fine to medium sand with gravel and occasional cobbles (loose, moist) (sod zone)
						SM	Dark brown silty fine to medium sand with gravel and occasional cobbles, (loose, moist)
5				8	■	SM	Dark brownish-gray silty fine sand (very loose, moist)
10				47	■	GP	Dark brownish-gray sandy fine to coarse gravel with occasional cobbles and a trace of silt (dense, wet)
15	%F			50/5"	□		
				100/5"	⊠		
20	GS			90/6"	⊠		
						GW GM	Brown fine to coarse gravel with silt, sand and occasional cobbles, (dense, wet)
25	GS			81/6"	⊠		
						GM	Gray silty, sandy fine to coarse gravel with occasional cobbles (dense, wet)
30	%F			57/6"	□		
				50/5"	■		
35				50/5"	⊠		
						SP SM	Gray medium sand with silt and occasional gravel (dense, wet)
40	%F			53/6"	■		



Boring completed at 40 feet on 8/31/90

Ground water encountered at approximately 12 feet depth at time of drilling and on 9/6/91

2-inch diameter monitor well installed to 25 feet on 8/31/90

Note: See Figure A-2 for explanation of symbols

:QMD:DJC:CDG 1/22/91

36-115-B06

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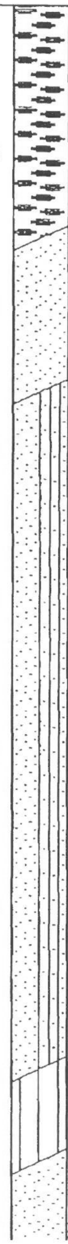
Log of Boring

Figure A-6

BORING B-4

TEST DATA

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow-Count	Samples	Group Symbol	DESCRIPTION
						Surface Elevation (ft.): 674.0
					GP	Gray fine to coarse gravel with coarse sand and occasional cobbles (medium dense, wet)
			50/5"			
					SP	Dark gray fine to coarse sand with gravel, occasional cobbles and a trace of silt (dense, wet)
			50/5"			
					SP SM	Gray gravelly fine to coarse sand with silt and occasional cobbles (dense, wet)
			50/6"			
			60/6"			
			62/5"			
			60/5"			
			50/4"		ML	Gray silt with sand and gravel (hard, wet)
					SP	Gray fine to coarse sand with gravel, occasional cobbles and a trace of silt (dense, wet)
			75/2"			



DEPTH IN FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40

Note: See Figure A-2 for explanation of symbols

86-115-B06 :GMD:DJC:CDG 10/16/90

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Log of Boring

Figure A-7

BORING B-4
(Continued)

TEST DATA

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow-Count	Samples	Group Symbol	DESCRIPTION
			100/4"	⊠		
			50/4"	⊠		Boring completed at 50 feet on 9/4/90 Ground water encountered at approximately 3 feet depth at time of drilling

DEPTH IN FEET

40 45 50 55 60 65 70 75 80

Note: See Figure A-2 for explanation of symbols

0186-115-B06 :GMD:DJC:CDG 10/16/90

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Log of Boring

Figure A- 8

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		TEST PIT 1
		Approximate elevation: 690 feet
0.0 - 0.5		4 inches minus rock spalls
0.5 - 2.0		12 inches minus rock spalls
2.0 - 4.0	SP	Dark grayish-brown gravelly fine to coarse sand with occasional cobbles, boulders and a trace of silt (medium dense, moist)
4.0 - 8.0	SP-SM	Gray gravelly fine to coarse sand with silt, occasional cobbles and small boulders (medium dense, moist)
8.0 - 10.0	SP	Dark grayish-brown gravelly fine to coarse sand with occasional cobbles, small boulders and a trace of silt (medium dense, moist)
		Test pit completed at 10.0 feet on 9/14/90
		No ground water seepage encountered
		Disturbed sample obtained at 2.5 feet
		TEST PIT 2
		Approximate elevation: 689 feet
0.0 - 0.5	GP	Gray fine to coarse gravel with sand (medium dense, moist)
0.5 - 1.5		12 inch minus rock spalls
1.5 - 10.5	SP-SM	Brown gravelly fine to medium sand with silt, occasional cobbles and small boulders (loose, moist)
		Test pit completed at 10.5 feet on 9/14/90
		No ground water seepaga encountered
		Disturbed sample obtained at 10.0 feet
		Large boulder encountered at 10.5 feet

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-9

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		<u>TEST PIT 3</u>
		Approximate elevation: 685 feet
0.0 - 0.5	SP-SM	Brown fine to medium sand with silt, gravel and roots (loose, moist) (sod zone)
0.5 - 6.0	GP	Dark grayish-brown sandy fine to coarse gravel with occasional cobbles, small boulders and roots to 2 feet in depth (loose, moist) (fill)
6.0 - 8.0	GP	Dark grayish-brown sandy fine to coarse gravel with occasional cobbles and small boulders
		Test pit completed at 8.0 feet on 9/14/90
		No ground water seepage encountered
		Large boulder encountered at 8.0 feet
		<u>TEST PIT 4</u>
		Approximate elevation: 688 feet
0.0 - 0.5	SP-SM	Brown fine to medium sand with silt and gravel (loose, moist) (sod zone)
0.5 - 8.0	SP	Dark gray gravelly medium to coarse sand (loose, moist) (fill)
		Test pit completed at 8.0 feet on 9/14/90
		No ground water seepage encountered
		Moderate caving below 5.0 feet

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-10

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		<u>TEST PIT 5</u>
		Approximate elevation: 683 feet
0.0 - 0.5	SP-SM	Brown fine to medium sand with silt and gravel (loose, moist) (sod zone)
0.5 - 7.5	GP	Dark gray sandy fine to coarse gravel with occasional cobbles, small boulders and a trace of silt (loose, moist) (fill)
		Test pit completed at 7.5 feet on 9/14/90
		No ground water seepage encountered
		Wood debris at 4.0 feet
		Moderate caving below 5.0 feet
		<u>TEST PIT 6</u>
		Approximate elevation: 687 feet
0.0 - 0.5	SP-SM	Brown fine to medium sand with silt, gravel and roots (loose, moist) (sod zone)
0.5 - 1.0		12 inches minus rock spalls
1.0 - 6.0	GP	Dark gray sandy fine to coarse gravel with occasional cobbles, small boulders and a trace of silt (loose, moist) (fill)
6.0 - 10.0	SP	Dark gray gravelly fine to coarse sand with occasional cobbles, small boulders and a trace of silt (loose, moist) (fill)
		Test pit completed at 10.0 feet on 9/14/90
		No ground water seepage encountered
		Large boulder encountered at 10.0 feet
		<u>TEST PIT 7</u>
		Approximate elevation: 685 feet
0.0 - 5.0	SP	Gray fine to medium sand with gravel, occasional cobbles, small bouldars and roots to 2.0 feet (loose, moist) (fill)
5.0 - 8.0	SP	Gray fine to medium sand with occasional gravel (loose, moist)
		Test pit completed at 8.0 feet on 9/14/90
		No ground water seepage encountered

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-11

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		<u>TEST PIT 8</u>
		Approximate elevation: 682 feet
0.0 - 3.5	SP	Gray gravelly fine to medium sand with occasional cobbles and roots to 2 feet (loose, moist) (fill)
3.5 - 6.0	SP	Gray gravelly fine to coarse sand with occasional cobbles, small boulders and a trace of silt (loose, moist)
		Test pit completed at 6.0 feet on 9/14/90
		No ground water seepage encountered

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-12

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		<u>TEST PIT TP-9</u>
0.0 - 11.5	GW	Dark brown fine to coarse gravel with fine to coarse sand, numerous cobbles to 12 inches (15 to 20 percent by weight), occasional boulders to 24 inches (5 to 10 percent by weight), tree roots and limbs to 4 inches in diameter, occasional concrete pieces (loose to medium dense, moist to wet)
11.5 - 13.0	GW-GM	Gray fine to coarse gravel with silt, fine to coarse sand and occasional cobbles to 12 inches (medium dense, to dense, wet) (Osceola mudflow)
		Test pit completed at 13.0 feet on 08/02/91
		Gravel, cobbles and boulders are subrounded
		Ground surface covered with layer of boulders to 18 inches
		Test pit walls sloughing, difficult to obtain sample of mudflow
		<u>TEST PIT TP-10</u>
		Water depth: 1.5 feet at test pit
0.0 - 12.0	GW	Dark brown fine to coarse gravel with fine to coarse sand, numerous cobbles to 6 inches (10 to 15 percent by weight), occasional cobbles to 12 inches (less than 5 percent by weight), occasional boulders to 24 inches (less than 1 percent by weight), numerous wood pieces, tree roots and limbs to 6 inches in diameter (loose to medium dense, wet)
		Test pit completed at 12.0 feet on 08/02/91
		Refusal on large obstruction at 12 feet (slab or boulder), moved test pit 15 feet downstream (Test Pit 2A)
		Gravel, cobbles and boulders are subrounded

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-13

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		<u>TEST PIT TP-11</u>
		Water depth: 1.5 feet at test pit
0.0 - 16.0	GW	Dark brown fine to coarse gravel with fine to coarse sand, numerous cobbles to 6 inches (10 to 15 percent by weight), occasional cobbles to 12 inches (less than 1 percent by weight), few boulders to 14 inches (less than 1 percent by weight), numerous tree roots and limbs to 6 inches in diameter, occasional metal pieces (loose to medium dense, wet)
		Test pit completed at 16.0 feet on 08/02/91
		Caving severely, digging stopped at 16-foot depth
		Gravel, cobbles and boulders are subrounded
		<u>TEST PIT TP-12</u>
		Water depth: 1 foot at test pit
0.0 - 15.0	GW	Dark brown fine to coarse gravel with fine to coarse sand, numerous cobbles to 6 inches (10 to 15 percent by weight), no boulders, occasional tree roots and limbs to 3 inches in diameter (loose to medium dense, wet)
		Test pit completed at 15.0 feet on 08/02/91
		Severe caving stopped digging at 15.0 feet
		Gravel and cobbles are subrounded
		<u>TEST PIT TP-13</u>
		Ground surface: 1 foot higher than dam apron
0.0 - 7.0	GW	3-inch to 12-inch cobbles in dark brown fine to coarse sand and fine to coarse gravel matrix, boulders to 24 inches (20 to 25 percent by weight) (loose to medium dense, moist to wet)
		Test pit completed at 7.0 feet on 08/02/91
		Caving severely at 7.0 feet, can't keep hole open below 7.0 feet depth
		Gravel, cobbles and boulders are subrounded
		Surface covered by boulders to 24 inches
		Water observed at 6.5 feet

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-14

GEI 110-103

LOG OF HAND HOLES

Hole No.	Depth Interval (inches)	U.S.E.	Soil Description
H-1	0 - 6	SM	Silty fine sand with a trace of organic material
	6 - 18	GP-GM	Coarse gravel with soft and fine sand
			Hand hole completed on 4/16/92
			Soil samples obtained at 6 and 18 inches
			Hole did not fill with water
H-2	0 - 6	SP-SM	Fine sand with silt
	6 - 18	SM	Silty fine sand
	18 - 36	SM	Silty fine sand with fine to coarse gravel
			Hand hole completed on 4/16/92
			Soil samples obtained at 6, 18 and 36 inches
			Hole filled slowly with water
H-3	0 - 12	GP	Coarse gravel with medium sand
			Hand hole completed on 4/16/92
			Soil sample obtained at 12 inches
			Hole filled rapidly with water
H-4	0 - 12	GP	Medium sandy fine to coarse gravel with occasional fine to coarse sand
			Hand hole completed on 4/16/92
			Soil samples obtained at 12 inches
			Hole filled rapidly with water

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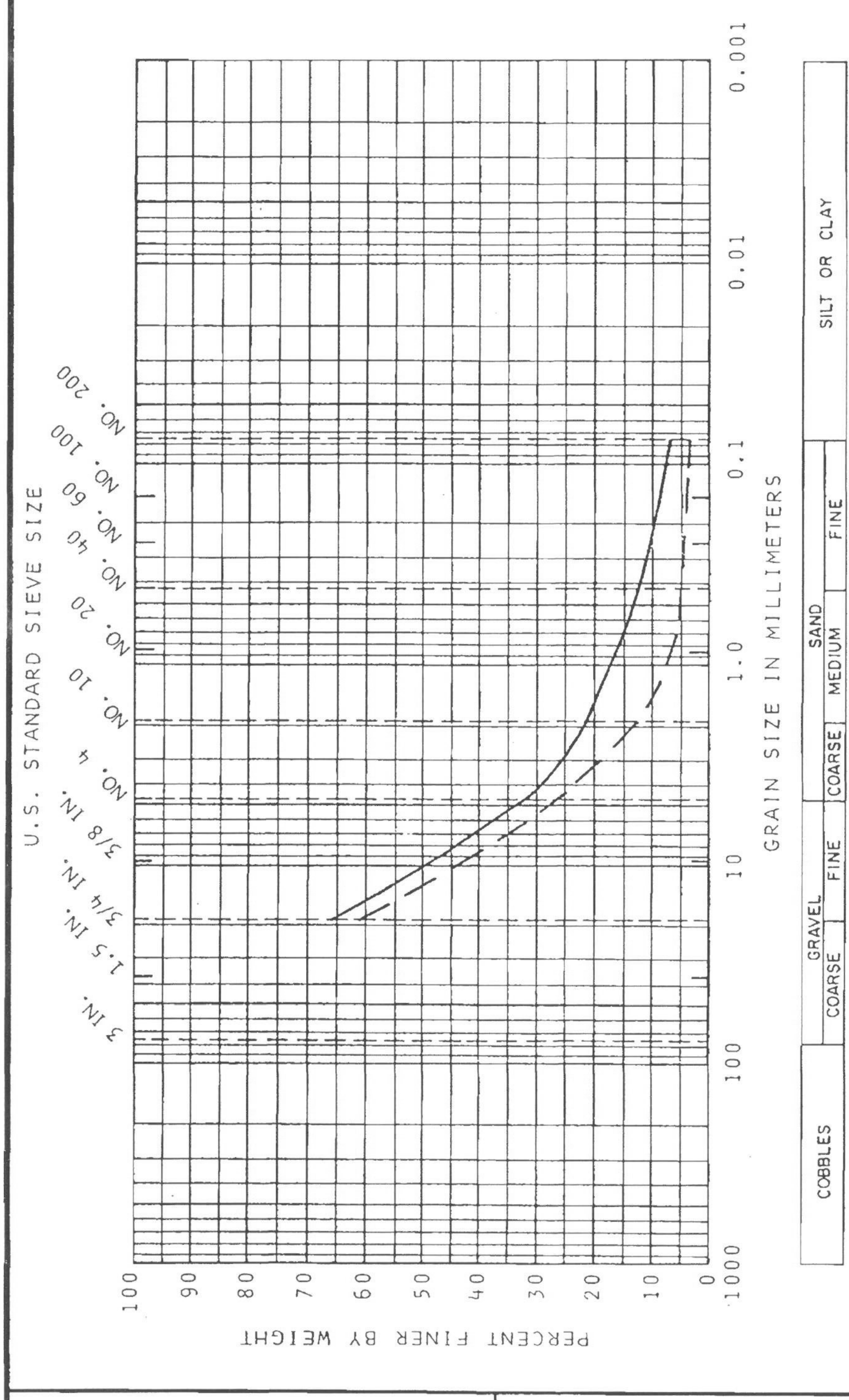
LOG OF HAND HOLES

FIGURE A-15

GE [illegible]0-88 0186-115-B06 DJC:BDH 10.17.90



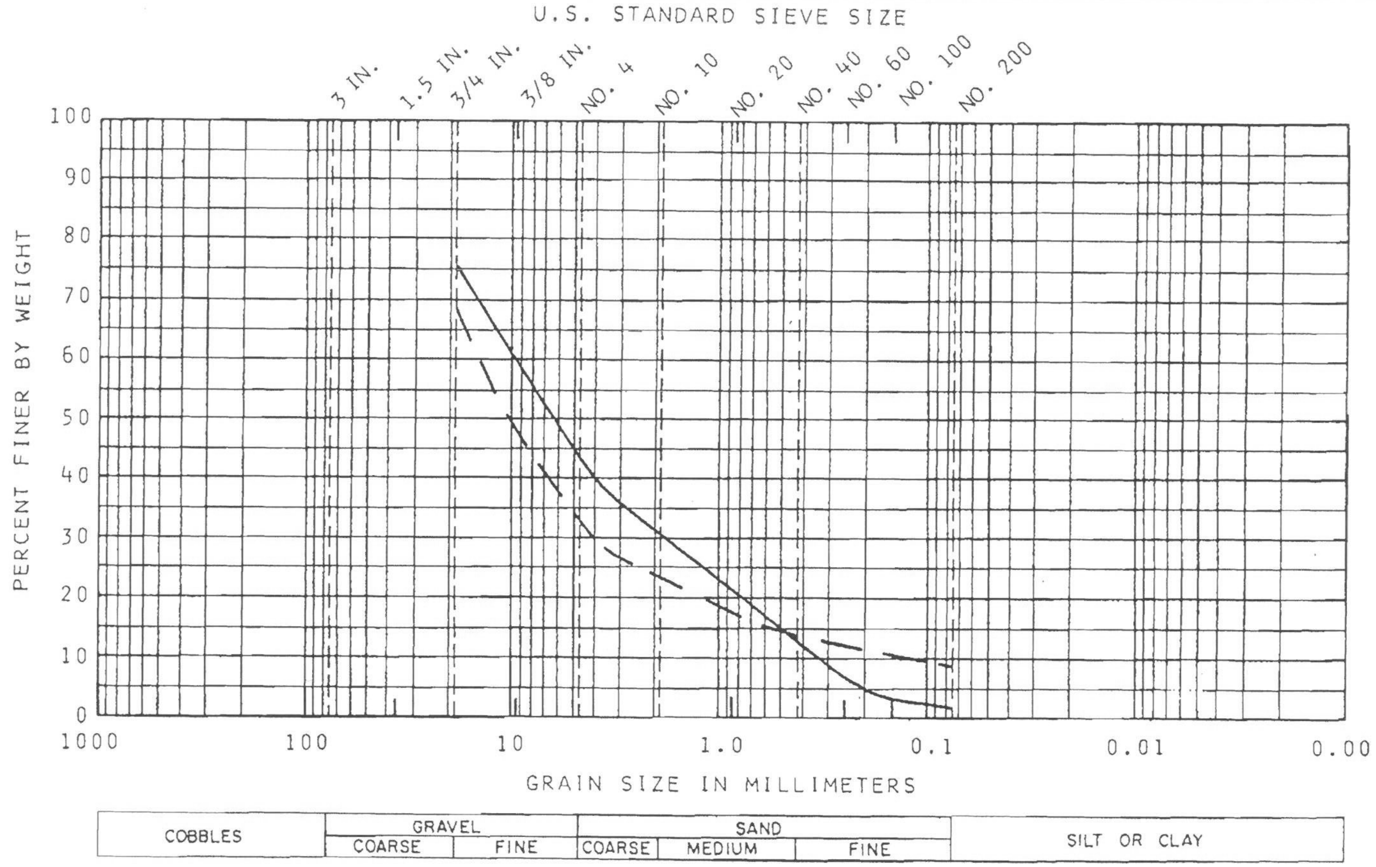
SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
	B-1 SAMPLE 1	7'	FINE TO COARSE GRAVEL WITH SILT AND SAND (GP-GM)
	B-1 SAMPLE 9	3.5'	FINE TO COARSE GRAVEL WITH SAND (GW)

GeoEngineers

GRADATION CURVES

FIGURE A-16

GE 0-88 0186-115-B06 DJC:BDH 10.17.90



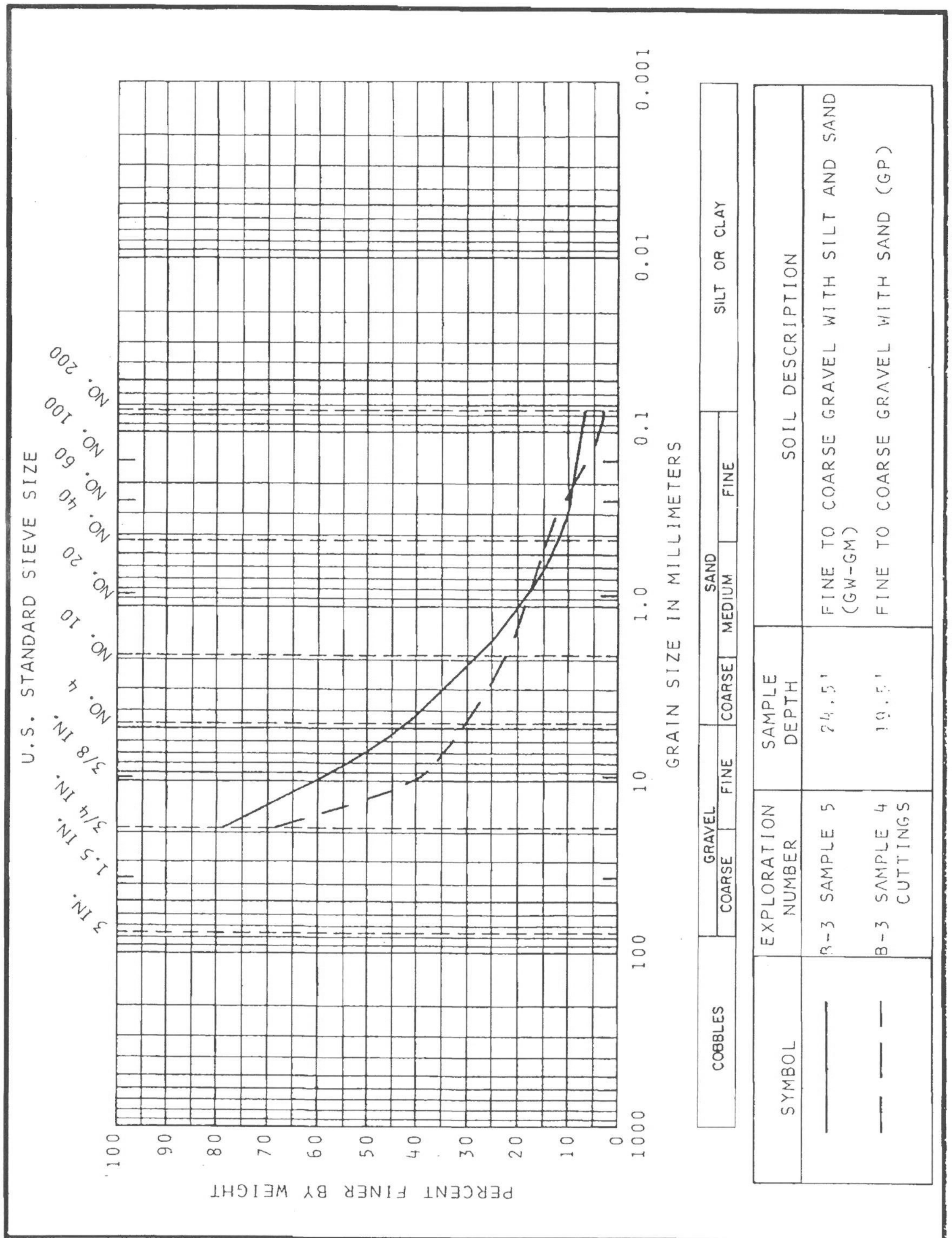
SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
————	B-2 SAMPLE 2	7'	FINE TO COARSE GRAVEL WITH SAND (GW)
— — —	B-2 SAMPLE 7	19.5'	FINE TO COARSE GRAVEL WITH SILT AND SAND (GP-GM)

GeoEngineers

GRADATION CURVES

FIGURE A-17

GE 0-88 0186-115-B06 DJC:BDH 10.17.90

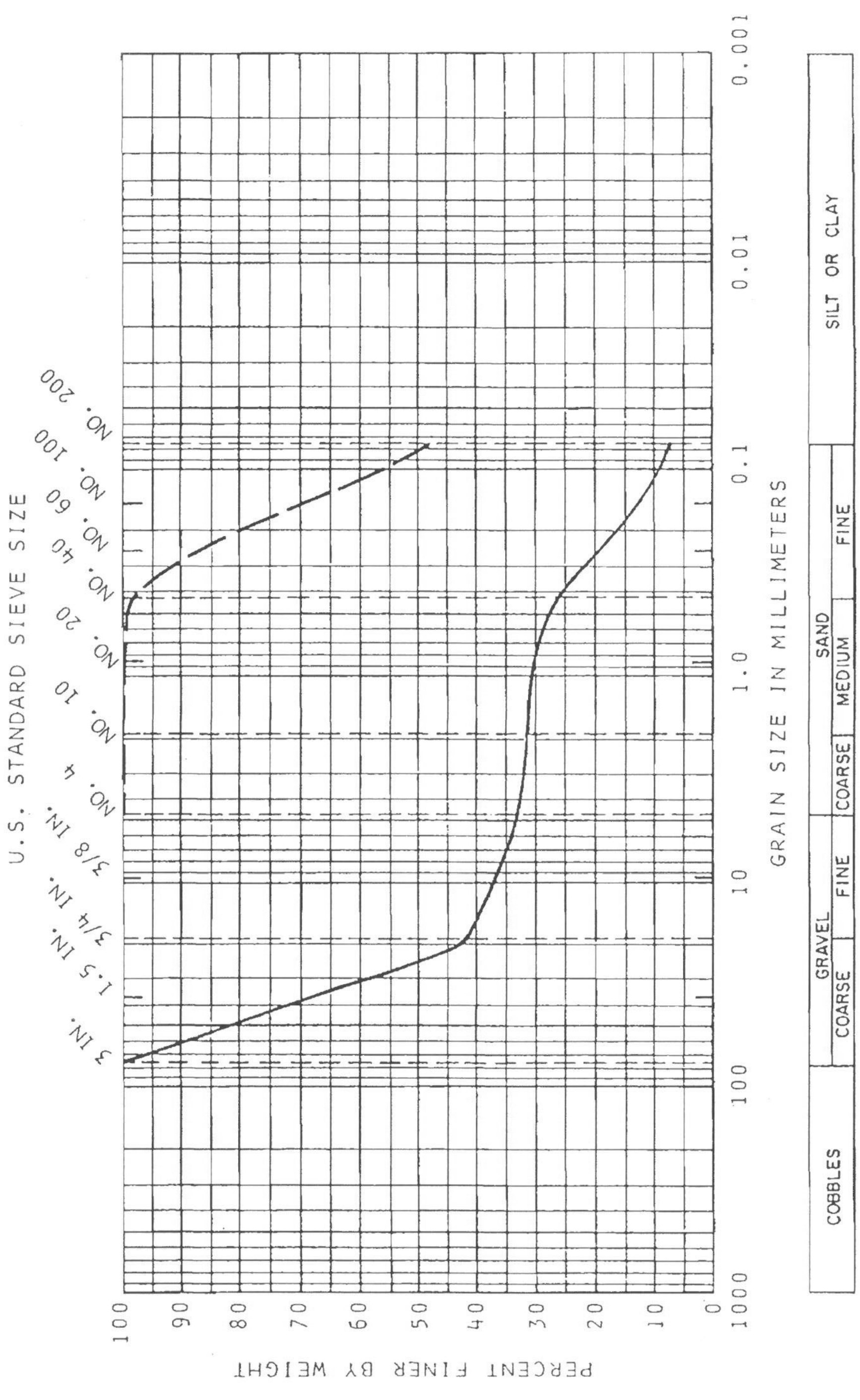


Geo Engineers

GRADATION CURVES

FIGURE A–18

0186-115-R06



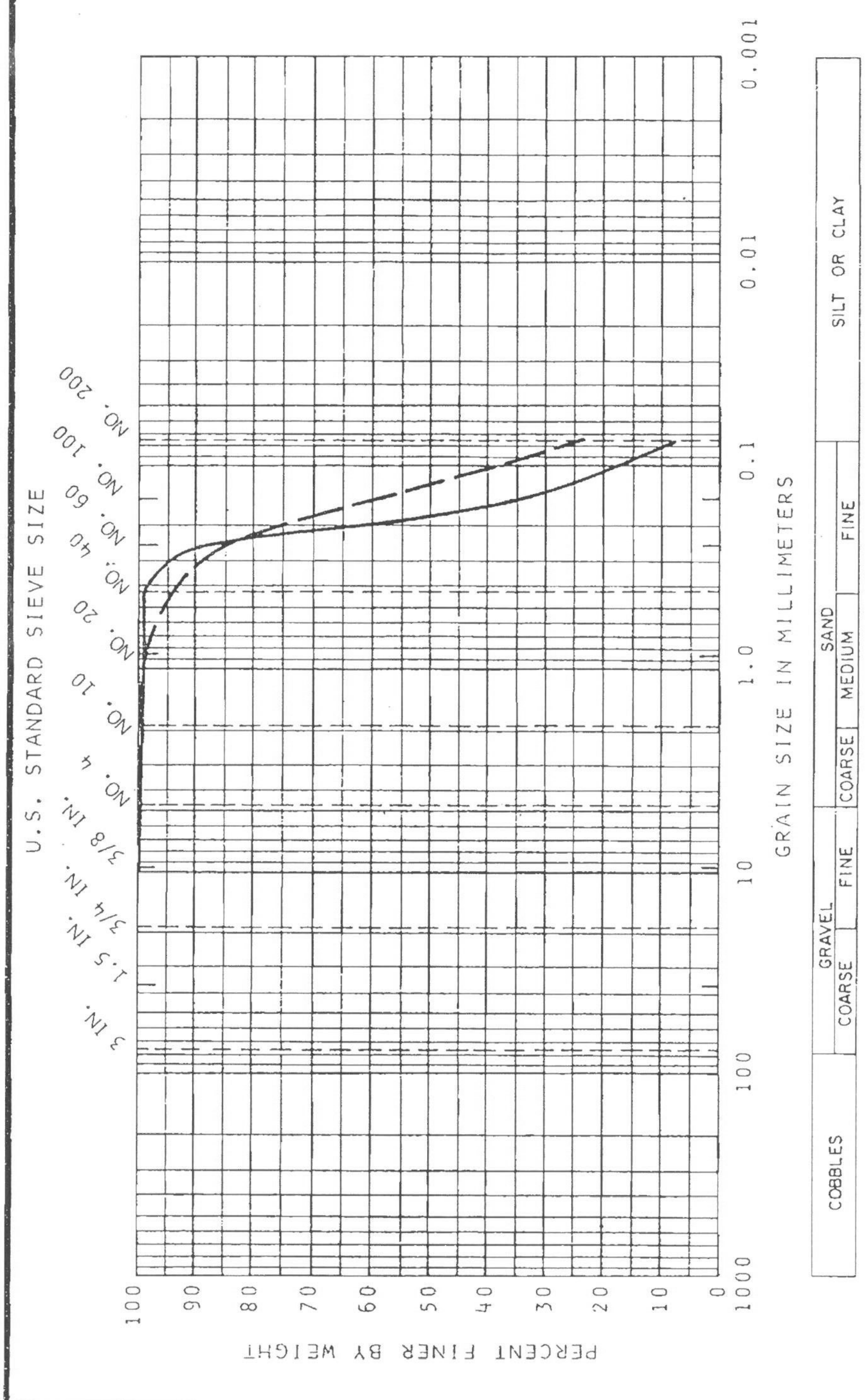
SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
———	H-1	18"	COARSE GRAVEL WITH SILT AND FINE SAND (GP-GM)
— — —	H-1	6"	SILTY FINE SAND WITH A TRACE OF ORGANIC MATERIAL (SM)

GE 0-88

Geo Engineers

GRADATION CURVES

FIGURE A-19



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
———	H-2	6"	FINE SAND WITH SILT (SP-SM)
— — —	H-2	18"	SILTY FINE SAND (SM)

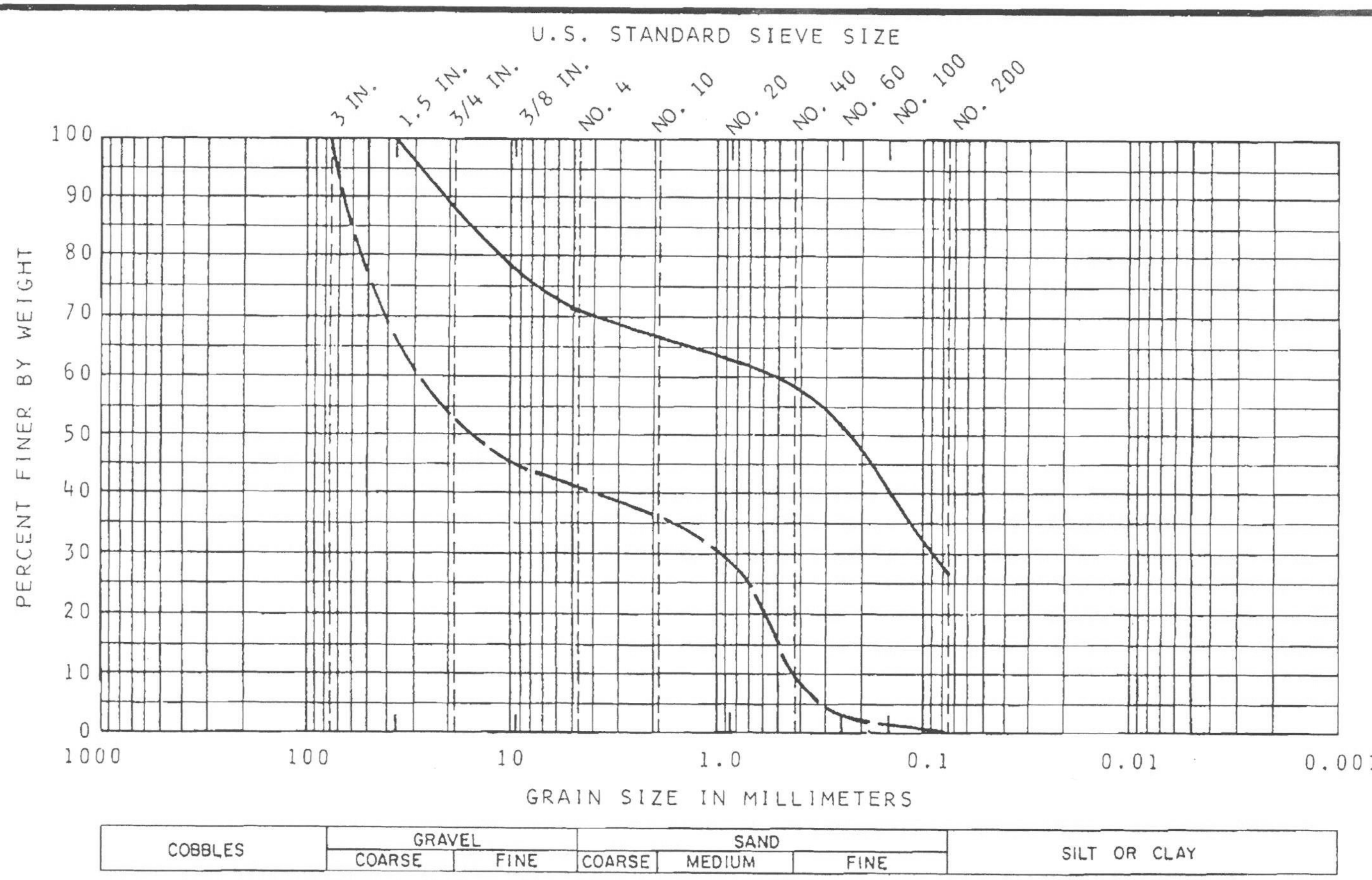
GeoEngineers

GRADATION CURVES

FIGURE A-20

GE[illegible]0-88

0186-115-R06



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
————	H-2	36"	SITLY FINE SAND WITH FINE TO COARSE GRAVEL (SM)
— — — —	H-3	12"	COARSE GRAVEL WITH MEDIUM SAND (GP)

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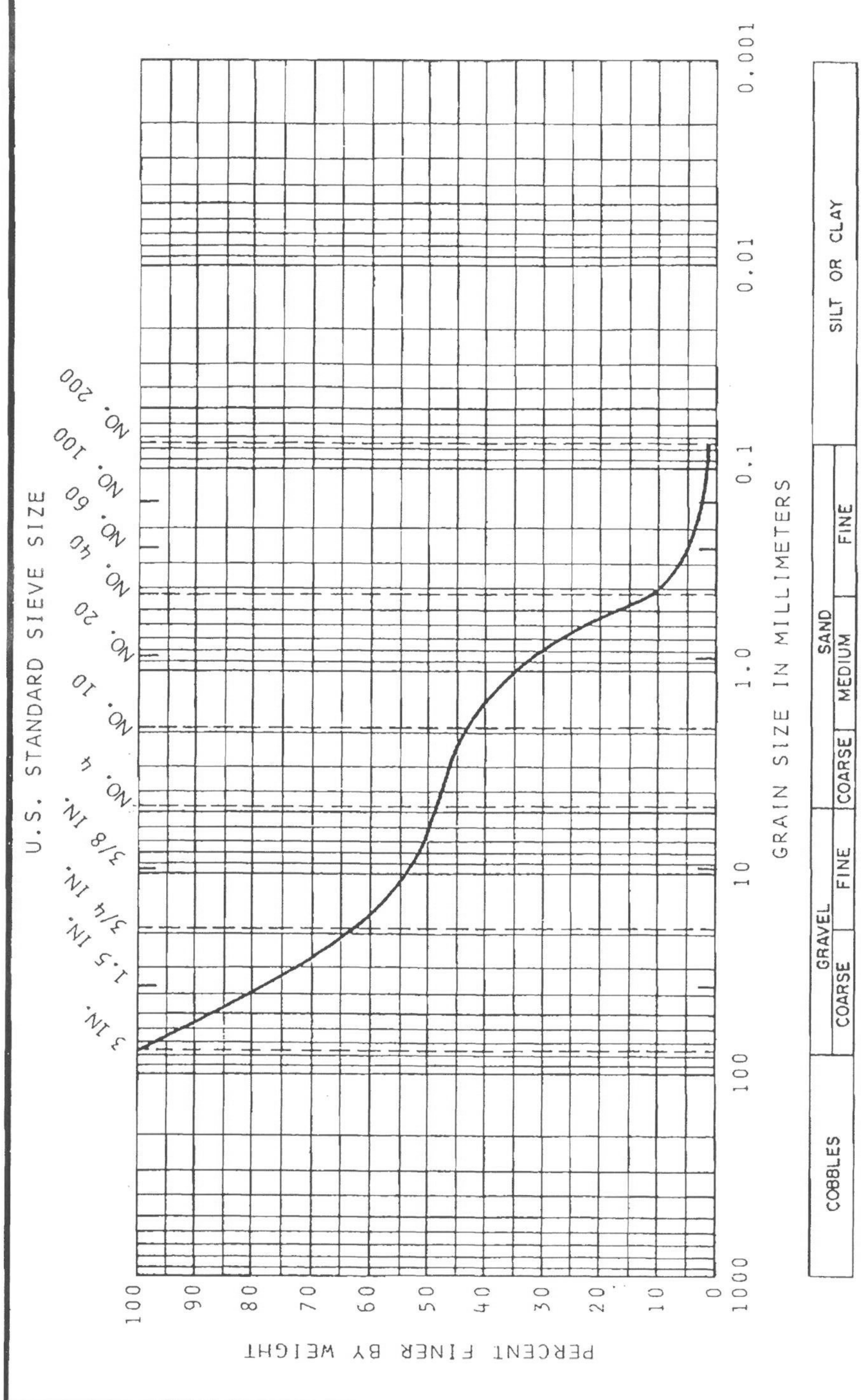
GRADATION CURVES

FIGURE A-21



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
——	H-4	12"	MEDIUM SANDY FINE TO COARSE GRAVEL WITH OCCASIONAL FINE TO COARSE SAND (GP)

0185-115-R06

GE 0-88

GeoEngineers

GRADATION CURVES

FIGURE A-22

PERCENT FINES DATA

Boring Number	Sample Depth (feet)	Sample Description	Percent Fines (%)
B-1	17.0	Fine to coarse gravel with a trace of silt (GP)	4.9
B-1	30.0	Fine to coarse gravel with sand (GW)	0.8
B-1	40.0	Fine to coarse gravel with silt and sand (GP-GM)	7.4
B-2	12.0	Fine to coarse gravel with silt and sand (GP-GM)	5.2
B-2	29.5	Fine to coarse gravel with sand and a trace of silt (GP)	4.4
B-2	39.5	Fine to coarse gravel with silt and sand (GP-GM)	7.5
B-3	15.0	Fine to coarse gravel with sand and a trace of silt (GP)	3.0
B-3	30.0	Silty fine to coarse gravel with sand (GM)	16.6
B-3	39.5	Fine to coarse gravel with silt and sand (GP-GM)	5.9

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PERCENT FINES DATA

FIGURE A-23

SLUG TEST RESULTS

Slug Test Number	Permeability (cm/sec)	
	Hvorslev's Method	Bower-Rice Method
1	1.2 x 10-2	1.0 x 10-2
2	3.4 x 10-2	2.4 x 10-2
3	8.2 x 10-3	6.4 x 10-3
4	2.6 x 10-2	1.7 x 10-2

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SLUG TEST RESULTS

FIGURE A-24

APPENDIX B

SIGMUND D. SCHWARZ
Consulting Geologist/Geophysicist

P.O. Box 82-917
Kenmore, WA 98028
(206) 823-5596

September 25, 1990
S86-90R

GeoEngineers, Inc.
2405 149th Avenue N.E.
Suite 105
Bellevue, Washington 98005

Att: Gordon Denby, PE

Re: Report of Geophysical Surveys, Puget Power White River Dam Reconstruction Project, Buckley, Washington

SUMMARY

Results of several geophysical surveys completed at this site indicate the area to be underlain by dense, relatively coarse grained alluvial and mudflow deposits. These deposits are indicated by their geophysical properties to be generally of uniform characteristics within the area explored and typical of the materials identified by the test borings. Bedrock occurs at relatively shallow depth several hundred feet north of the damsite area and deepens beyond the depth of exploration to the south beneath the river.

The primary objective of this work has been to characterize the nature of materials underlying the existing White River Dam to assist in the geotechnical aspects of design for the new structure and more specifically to identify anomalous zones where unexpected conditions might be encountered. The north abutment area has also been studied to develop information concerning potential leakage paths.

Geophysical surveys incorporating several complementary exploration methods have been completed at this site. Some elements of this work have been carried out under subcontract or by previous contract. These methods include seismic, electrical and electromagnetic techniques that are listed as follows:

1-GPR(ground penetrating radar)	Williamson and Associates
2-EM(electromagnetic)	Williamson and Associates
3-VES(vertical electrical soundings)	Schwarz
4-Overwater seismic refraction	Schwarz
5-Land seismic refraction	EBASCO Services (Nov. 1982)

The location and results of this survey are shown on the Geophysical Exploration Plan, Fig. 1 and Composite Geophysical

Profile A-A', Fig. 2 which includes the interpreted result of all geophysical surveys.

The damsite area was explored with 6 VES soundings, GPR, EM and overwater seismic refraction. The overwater seismic refraction survey was confused by the presence of high velocity concrete in the foundation of the existing dam and was therefore ineffective.

The north abutment area was explored by GPR and EM methods together with a land seismic refraction survey completed by EBASCO Services for Puget Sound Power and Light Corporation in 1982.

Interpreted results of the geophysical surveys are shown on the Composite Geophysical Profile A-A', Fig. 2. VES and GPR data are the most effective for delineating overburden stratigraphy in the damsite area. The VES data is depicted on the profile as a matrix of calculated electrical resistivity values derived from the six VES soundings and expressed as electrical resistivity in terms of ohm metres. Over this is superimposed GPR reflecting boundaries and the average seismic velocity. The GPR and EM survey was extended into the north abutment area to supplement the seismic refraction data presented in the EBASCO report. The agreement between data obtained by various geophysical methods and the borings is generally good.

Based upon these data, it appears that the site area is underlain by a fairly thin mantle of coarse grained recent alluvium over volcanic mudflow. The mudflow deposit is indicated by the seismic, VES and GPR data to be dense, basically hetrogeneous and very crudely stratified with a gentle southerly dip. The electrical and seismic characteristics of the mudflow are typical of those observed in the Osceola at other locations in the area.

The EBASCO seismic survey indicates bedrock to occur within 25 to 30 feet of the ground surface near the far north end of the geophysical profile and to be deepening to the south. These data indicate bedrock to be deeper than 60 to 70 feet in the damsite area.

The data presented herein is interpretive in nature and subject to revision on the basis of additional site specific information that may become available.

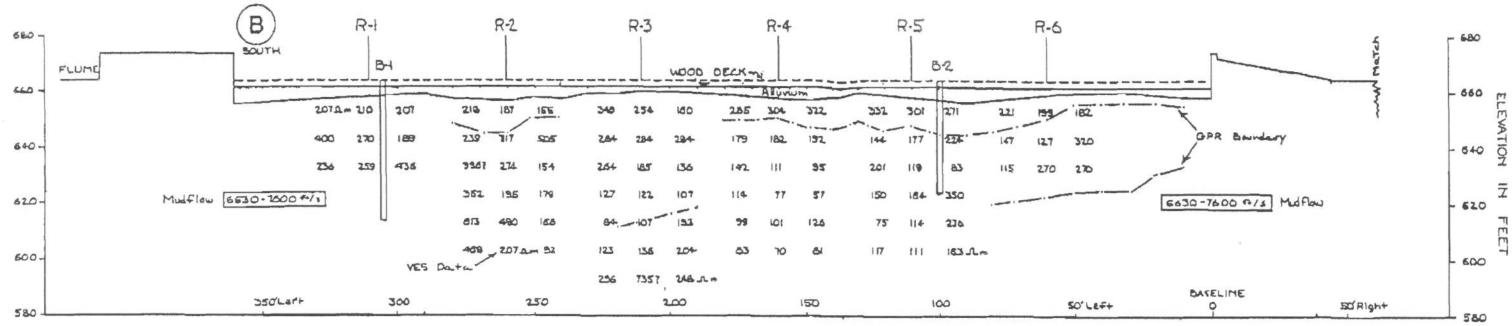
Please do not hesitate to contact me if you have any questions concerning this report or if I may be of further service.

Respectfully submitted,

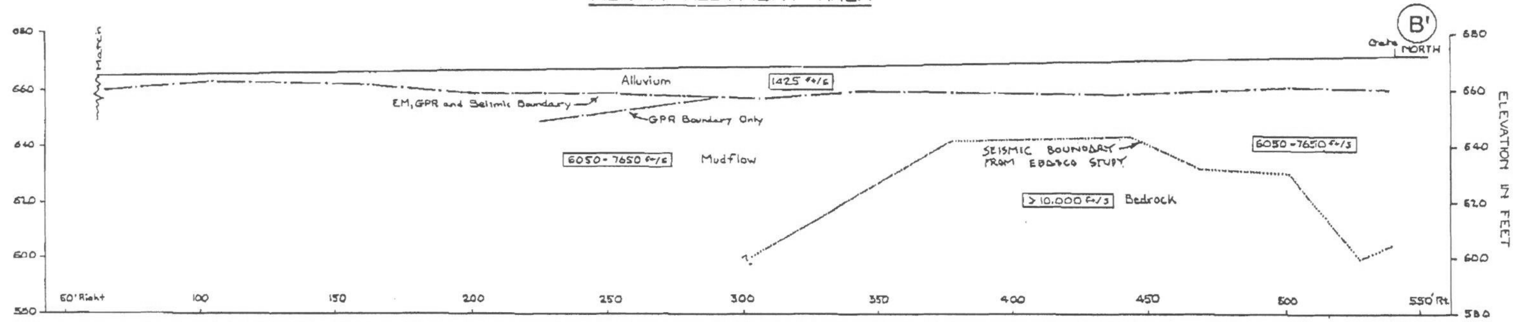
Sigmund D. Schwarz

Encl: Fig. 1 Exploration Plan, Fig. 2 Composite Profile A-A'

DAMSITE AREA



NORTH ABUTMENT AREA



EXPLANATION:

GPR	GROUND PENETRATING RADAR
R-1	VERTICAL ELECTRICAL SOUNDING (VES) LOCATION AND NUMBER
284	VES RESISTIVITY DATA IN MICRO OHMS
B-1,2	GEI BORINGS (THIS STUDY, REFER TO FIGURE 3 AND APPENDIX A)
-------	EBASCO STUDY (1982) SEISMIC BOUNDARY
1425 ft/s	CALCULATED VELOCITY FROM SEISMIC STUDIES

NOTE: The information presented herein is interpretive in nature and subject to revision on the basis of additional site-specific information that may become available.

REFER TO FIGURE 2 FOR PROFILE LOCATION

0 40 80
SCALE IN FEET

WHITE RIVER DAM

COMPOSITE GEOPHYSICAL PROFILE B-B'

S.O. Schwarz, Consultant 9-25-90

REVISED BY GEOENGINEERS 10/12/90

FIGURE B-1

TECHNICAL MEMORANDUM NO. 16

SUBJECT:	**FINAL DESIGN REPORT** **GEOTECHNICAL ENGINEERING SERVICES** **PROPOSED DIVERSION DAM REPLACEMENT** **WHITE RIVER HYDROELECTRIC PROJECT** **BUCKLEY, WASHINGTON**
PREPARED FOR:	Puget Power Sound & Light Company HDR Engineering, Inc.
PREPARED BY:	Daniel W. Mageau - GeoEngineers, Inc.
APPROVED BY:	Gordon M. Denby - GeoEngineers, Inc.
PROJECT NAME:	White River Diversion Dam
PROJECT NO.	GEI File No. 0186-227-R06
DATE:	May 2, 1994 (FINAL)

This report presents the results of geotechnical engineering services completed for the final design of the proposed diversion dam replacement for the White River Hydroelectric Project. The site is located on the White River approximately 1/2 mile east of SR 410 near Buckley, Washington, as shown on the Vicinity Map and Site Plan, Figure 1.

The conclusions and recommendations presented herein supersede those in our preliminary report dated May 4, 1992 (Technical Memorandum No. 13, GEI File No. 0186-115-R06) and the draft final report date November 30, 1993 (Technical Memorandum No. 16). The May 4 preliminary report was prepared for use by HDR in the development of preliminary plans for the project. This final design report includes an updated description of the project, the results of additional explorations and new field test programs. It also includes design level geotechnical engineering recommendations for major dam components as well as sections that address construction considerations.

Our understanding of the project is based on discussions and meetings with Mr. Bob King of HDR and Mr. Mike Blanchette of Puget (Puget Sound Power & Light Company), plans by HDR entitled "P.S.P.& L. Co., White River Hydroelectric Project, FERC Project No. 2494, Diversion Dam Rebuild, Drawings 1 through 23," dated December 20, 1993 and our experience with other work for Puget at the White River Hydroelectric Project.

CONTENTS

CONTENTS (continued)

CONTENTS (continued)

PROJECT DESCRIPTION

The White River Project is an existing hydroelectric facility that consists of a diversion dam and intake structure; an 8-mile-long series of flumes, canals, and basins; a set of fish screens; a storage reservoir (Lake Tapps); an intake tunnel; a forebay well; four penstocks; and a four-unit powerhouse. It began operation in 1911 and is currently rated at 63.4 MW. The diversion dam is located approximately 1 mile north-northeast of Buckley, Washington. A vicinity map and a site plan showing the diversion dam and intake project areas are presented in Figure 1. A site plan showing more detail of the existing dam site is presented in Figure 2.

The existing diversion dam consists of a 352-foot-long by approximately 4-foot-thick concrete and rock-filled timber crib structure. Wood flashboards extend 7 feet above the crib structure. The water level behind the dam is maintained at Elevation 671 feet. To facilitate flashboard replacement and removal, a cable tramway is suspended over the dam. The dam is protected on both upstream and downstream faces by timber aprons. A six-foot-deep concrete cutoff wall extending about 8 feet below original riverbed underlies the length of the dam. These cutoffs are located near the upstream and downstream edges of the dam. The dam abutments consist of thick unreinforced vertical concrete walls.

The proposed project consists of replacing the existing timber crib dam and adding intakes, dikes, headgate buildings and fishways. Major components of the replacement dam project are shown in Figure 3. The replacement dam will consist of two radial gates (16- and 35-foot-wide), two 50-foot-wide rubber weirs, and six 20-foot-wide, removable, fixed crest concrete panels. The proposed design calls for the right (northern) two-thirds of the existing dam foundation to remain in place and be used as foundation support for the weirs and fixed panel portions of the replacement dam. The radial gates will be supported on a thick concrete mat bearing on native soil. A downstream end wall will be constructed to protect the dam foundation from toe scour.

The replacement dam will have provisions to regulate flows, allow for fisheries flow requirements, and prevent bedload sediment from entering the intake. As part of the project, the existing intake and the headgates will be modified. Auxiliary fish way structures constructed of concrete walls supported on a concrete mat are also planned. A maintenance building, control building and equipment building will be constructed in the vicinity on the intake.

Construction of the replacement dam will be completed in two phases. Phase I includes the fixed crest concrete panels and one rubber weir. Phase II includes one rubber weir, the two radial gates, and the modification to the intake and headgates. Phase I requires construction of an approximately 500-foot-long cofferdam upstream and downstream of the existing dam. Phase II construction will require an approximately 400-foot-long cofferdam upstream and downstream of the dam. A temporary diversion channel constructed of driven sheet piles will be required near the intake during Phase II construction, as shown in Figure 3.

Modifications to the existing access road along the right bank of the river upstream of the dam are planned to reduce the potential for flooding near the existing fish hatchery. From the right abutment to about 500 feet upstream of the dam, the existing access road will be raised by 2 to 5 feet. Several studies have been completed by others for previous projects at and near the dam site. Aerial photographs and historical construction photographs and records are also available. A list of references reviewed for this project is summarized at the end of the text.

SCOPE

The purpose of our services is to provide geotechnical design information to HDR for use in developing design plans and specifications for the proposed diversion dam replacement. Our specific scope of services for this final design study includes the following:

FIELD AND LABORATORY PROGRAMS

1. Complete pile driving tests at two locations in the river.
2. Excavate near-surface soil at the two pile driving test locations and at two locations along the proposed temporary diversion channel alignment.
3. Complete a large test excavation to evaluate seepage conditions on the right bank.
4. Cut holes in the surficial timber planks and core the underlying concrete at three locations along the downstream apron of the existing dam. Evaluate the condition of the wood in the dam and the soils immediately below the dam at the hole locations.
5. Drill two additional borings through the downstream apron of the dam.
6. Install a 2-inch-diameter monitoring well (MW-1) in one of the borings and a 10-inch-diameter pump test well (TW-1) in the other boring.
7. Complete a ground water pump test in MW-1 to evaluate soil permeability below the dam.
8. Complete laboratory index tests on selected soil samples. Perform unconfined compression tests on two of the concrete core samples.

SITE AND SOIL CONDITIONS

1. Review results from previous exploration programs.
2. Present information regarding existing site conditions and bank stability contained in our May 4 preliminary report.
3. Review results from field and laboratory programs completed for this study.
4. Summarize soil conditions below the dam and in other areas of the project where new structures are planned.
5. Prepare a profile and cross section showing soil conditions below the dam.

TEMPORARY DIVERSION AND DEWATERING

1. Develop recommendations for the design and construction of earth cofferdams for various locations within the dam replacement site.
2. Present a discussion of dewatering considerations including anticipated soil permeability values and appropriate dewatering systems.
3. Develop recommendations for temporary excavations made in the river.
4. Develop recommendations for the construction of a temporary diversion channel using driven sheet piles.

EXISTING DAM STRUCTURE

1. Evaluate results from field and testing programs.
2. Develop conclusions regarding the integrity of the existing dam and subgrade and the feasibility of reusing the existing dam as foundation support for the fixed panel and rubber weir portions of the replacement dam.

FOUNDATION DESIGN RECOMMENDATIONS

1. Develop recommendations for the support of the radial gate section, including allowable soil bearing pressures as a function of settlement.
2. Develop recommendations for the support of the maintenance, control and equipment buildings.
3. Present recommendations for the support of other dam structures, including fish ways, access ramps, and upstream aprons.
4. Present recommendations for permanent excavations required in the maintenance building area.
5. Present recommendations for right bank dike design and construction.
6. Discuss river channel and river bank erosion protection.

SEEPAGE CONSIDERATIONS

1. Refine estimates of ground water seepage below the proposed dam based on new permeability data.
2. Discuss piping potential below the existing dam.
3. Present recommendations for the design and construction of a downstream end wall.

GEOLOGIC HISTORY

The dam site is located in a geologic region characterized by thin alluvial deposits overlying Osceola Mudflow deposits up to 70 feet thick in some areas. The Osceola Mudflow blanketed an extensive portion of the eastern Puget Sound Lowland approximately 3,700 years ago. The mudflow originated on the north flank of Mount Rainier, flowed down the White River Valley, and covered a wide area, including present-day Buckley, with several tens of feet of sediment – the mudflow sediment typically consists of cobbly, silty sand and gravel with cobbles and occasional boulders. The mudflow occurred in a series of separate flows in between and after which alluvial soils consisting of sand and gravel with cobbles and boulders were deposited by the White River. Moreover, water flow over and through the surface layer of the exposed mudflow deposits probably washed the silt portion out of the soil. This process resulted in a relatively heterogeneous mix of alluvium and mudflow deposits with less fines and siltier mudflow deposits below about 10 feet. Glacially deposited sands and gravels typically underlie the mudflow deposits.

Soils exposed within a 25- to 40-foot-high riverbank along the left bank (south side) of the White River, immediately upstream of the diversion dam, show approximately 10 feet of an alluvial deposit overlying Osceola Mudflow sediments, indicating that the river has incised through the mudflow deposits at this location.

SURFACE CONDITIONS

RIGHT BANK

The White River Fish Hatchery is located about 100 feet north of the right bank of the river, just upstream of the dam (refer to Figure 1). A gravel dike is situated along the right bank of the White River to protect the bank from erosion and the fish hatchery from flooding during periods of high water. The approximate location of the right bank dike is outlined in Figure 1.

The right bank dike is comprised of several sections which were constructed at different times. The original dike was constructed in 1911 and extends along the river to a point about 1,800 feet upstream of the dam. It is approximately 3 to 6 feet high (with respect to native ground surface north of the dike). This dike is covered with brush and trees ranging in diameter (at breast height) from 4 to 36 inches. It is protected on the river side by large boulders and concrete rubble up to 8 feet in size.

A newer portion of the dike constructed in 1968 extends from about 1,800 to 2,500 feet upstream of the diversion dam. It is approximately 8 to 12 feet high (with respect to the ground surface north of the dike). This portion of the dike is composed of sand and gravel and is protected by quarry rock typically ranging from 2 to 24 inches in size.

The right bank dike appears to be relatively stable at this time. Some loss of finer rock occurred prior to 1992 in unmaintained areas of the dike. According to King County maintenance personnel, new rock protection was placed over the dike near Station 23+00 where more significant loss of rock protection occurred during the 1989-90 winter high river flow. We understand that King County has repaired the dike every few years since 1968 on an as-needed basis. Repair of the dike protection involved dumping and spreading well-graded crushed quarry rock (obtained from the Enumclaw quarry located on Southeast 416th Street in Enumclaw, Washington) and spreading the material in a 1- to 3-foot thick lift. Maximum size of the quarry rock is about 2 feet.

The newer section of the right bank dike was raised 4 to 7 several feet in 1992 as part of the hatchery water intake construction project. New fill was placed between about Station 13+00 and 25+00. We understand that sand and gravel was used as fill material. Riprap several feet thick and up to 24 inches in size was placed over the entire face of the slope along the river.

The top of the dike ranges in elevation from approximately 673 feet near the dam to approximately 689 feet 2,500 feet upstream of the dam. The water surface elevations of the White River near these locations were approximately 671 and 678 feet, respectively, on September 14, 1990 based on hand level measurements.

An access road leading to the right abutment of the dam was constructed in the late 1980s. In addition to access for the dam, it also serves as a secondary dike to further protect flooding of the hatchery. The location of this access road is shown in Figure 1. The road appears to be composed of sand and gravel, based on evaluation of surface soils. It is protected on the river side by 12-inch-diameter quarry spalls. The road extends from the right abutment east approximately 800 feet.

The topography of the right bank of the river (north of the original dike) is relatively flat, except for the access road and rill pads placed for the hatchery, with elevations ranging from 665 feet near the fish hatchery to about 680 feet approximately 1/2 mile upstream of the diversion dam. Large sand and gravel bars are present near the right bank of the river, just upstream of the dam. The ground surface elevations of the sand bars typically range from 670 to 678 feet.

Most of the site upstream of the diversion dam is heavily forested with fir, cedar, and alder trees. Low undergrowth consists of grasses, nettles, ferns, and scattered blackberry bushes. Scattered open areas of grass and brush with scattered alder and maple trees are present along the eastern end of the right bank dike. The hatchery site is an open grassy area.

LEFT BANK

The intake structure and flume are located near the left abutment of the dam (refer to Figure 1). A 25- to 40-foot-high bluff forms the left bank of the river immediately upstream of the diversion dam. The ground surface above the bluff (south) is relatively flat and ranges in elevation from about 690 to 695 feet. The bluff extends approximately 900 feet upstream of the diversion dam. Further upstream, along the left bank of the river, the ground surface is several feet above the river level. The topography in this are a is relatively flat with ground surface elevations ranging from 675 to 685 feet.

RIVER EROSION AND DEPOSITION

A geotechnical engineer and an engineering geologist from our firm completed a field reconnaissance on September 18, 1990 to evaluate the existin g condition of the riverbanks for 1/2 mile upstream of the dam. We completed several other site reconnaissances subsequently and reviewed aerial photographs of the area taken in 1936, 1968, 1980 and 1985. Additional historical information regarding construction of the dike along the right bank was obtained from discussions with Puget engineers and King County maintenance personnel.

The results of our reconnaissance and aerial photograph review indicate that relatively little erosion of the left riverbank for 1/2 mile upstream of the dam has occurred since the diversion dam was constructed in 1911. From aerial photographs, it appears that the bluff along the left bank just upstream of the dam may have receded 5 to 10 feet since 1936 (an average of about 1 to 2 inches per year). In other areas of the river channel, the banks appear to have aggraded since 1936. Sediment accumulation is particularly evident along the right bank of the river just upstream of the dam. Sand and gravel bars on the order of 100 feet wide presently extend about 1,000 feet upstream of the dam. Only portions of these bars are evident in the 1936 photograph. In general, the river channel appears to have beco me narrower and more stable (i.e., less meandering) since about 1968. During our field reconnaissances, we did not encounter evidence of any significant erosion along the banks. Some minor sloughing along the steep bluff on the left bank was observed. Our experience with similar bluffs composed of mudflow deposits elsewhere in the White River indicates that although the river tends to meander along the valley, very little lateral erosion of the bluffs occurs.

The sand and gravel bars near the right bank of the river are currently covered with brush and small to moderate-sized trees. Near-surface soil conditions observed in hand holes excavated in these bars and in shallow erosion cuts along the river typically consist of 1/2 to 2 feet of fine to medium sand underlain by sandy gravel with cobbles in the downstream two-thirds of the bars. In the upstream one-third, the surface of the bars is predominantly covered with gravel and cobbles ranging from 1 to 10 inches in size. Some sand is mixed in with the gravel and cobbles. Based on visual observations of soil conditions encountered in the hand holes, we estimate the average percentage of cobbles in the upper 2 to 3 feet of the bars to be about 20 to 30 percent by volume. A description of the hand hole excavation methods as well as a log of the soils encountered are presented in Appendix A.

The vegetation along the right bank (north of the sand and gravel bars) is well established. The large rock and concrete bank protection placed on the older dike in the early 1 900s is surrounded by brush and trees. It appears that this area of the bank has not been subject to significant water flow or erosion for many years. The sizes of trees along this existing dike range from 4 inches to over 36 inches in diameter.

FIELD AND LABORATORY PROGRAMS

GENERAL

GeoEngineers completed a series of field programs at the dam site to supplement existing subsurface information obtained from our previous studies and to obtain new information to address construction issues. Results from previous field studies at this site are presented in our May 4, 1992 preliminary report. For this final design study we completed the following: sheet pile driving tests, test pit excavations in and near the river, concrete cores in the existing dam, two borings through the existing dam, a monitoring well and a pump test well, a ground water pump test and a large test excavation to evaluate seepage and excavation issues. We also completed laboratory tests on selected soil samples recovered from our borings and compression strength tests on two of the concrete cores taken from the existing dam. Details of each field program and the laboratory testing are presented in the following sections.

TEST PILE DRIVING

General

Sheet pile cofferdams were considered for temporary diversion of the river during construction of the replacement dam in an earlier design of the project. However, it was not known at that time whether it was feasible to install sheet piles through the cobbles and boulders anticipated at the site. The purpose of the sheet pile driving test program was to evaluate sheet pile drivability in the river bed material. Pile driving tests were completed in two general areas designated as location A and location B, as shown on the Site Plan, Figure 2.

Equipment

The pile test program was completed by Pile Contractors, Inc. of Issaquah, Washington on August 31 and September 1,1993. Two different sizes of vibratory hammers were used to drive the sheet piles: the smaller-sized Tunkers 60.05 vibratory hammer rated at 66 tons drive force at a maximum frequency of 1500 rpm (revolutions per minute) and a larger-sized MKT V-30 vibratory hammer rated at 160 tons drive force at a maximum frequency of 1600 rpm. A pair of 3/8-inch-thick Z-piles were used for the test program. The piles were generally driven with pile tips welded onto the bottom of the piles, except for the very first attempt (A-1) at location A (refer to Figure 4).

Results

Details of the pile driving records at each location are presented in Appendix C (Tables C-1 and C-2). Pile driving typically was extremely difficult with minimal penetrations being achieved. The sheet piles generally could not be driven plumb and bounced severely on underlying cobbles and boulders during driving. The bottoms of the piles were severely worn as a result of grinding against the underlying soils and the pile tips were curled as a result of pile driving. A brief summary of driving results at each test location is presented below.

Location A. The sheet piles were driven at three specific locations, designated as A-1, A-2 and A-3, as shown in Figure 4. Locations A-1 and A-2 were in the river. Location A-3 was on-shore (on the left bank of the river). No pile tips were used on the bottom of the sheet piles for the first attempt at location A-1. After three minutes of driving using the Tunkers 60.05 hammer, a penetration of only 2 feet into river bed material was achieved. The sheet piles were then removed and examined. The bottom 2 feet was severely worn. Pile tips were welded onto the bottom of the sheet piles for all subsequent pile driving tests. A maximum penetration of 41/2 feet into the river bed material was recorded for the second attempt at location A -1. This penetration took approximately 19 minutes. The depths of penetration at the A -2 and A -3 locations were only 21/2 and 3 feet, respectively. The larger MKT V -30 hammer was not used for any of the piles at location A because the "pile seating depth" necessary to mount the hammer on the pile could not be achieved.

Location B. Sheet piles were driven at six specific locations, designated as B-1 through B-6, as shown in Figure 5. The results at these six locations varied somewhat, but in general the driving was very difficult, particularly below a depth of about 5 to 6 feet. At the first attempt at location B-1, the sheet pile was driven to a depth of about 8 feet below the river bed. The pile appeared to bounce on a boulder or hard debris at this depth. At locations B -2 through B-4, the sheet piles were driven about 4 to 5 feet into the existing river bed soils within 2 to 3 minutes without having to excavate any river bed material. However, refusal was met at this depth at these test locations. After excavating the uppermost 3 to 5 feet of river bed material, the piles at locations B-5 and B-6 still encountered refusal on boulders at relatively shallow depths. The Tunkers 60.05 vibratory hammer generally was used, except at location B -1 where the MKT V - 30 hammer was used to drive the sheet piles during the second attempt. A maximum penetration of 41/4 feet was recorded for the second attempt at location B -1 after driving for 81/2 minutes using the MKT V-30 hammer.

Subsequent to our pile driving test program, information was obtained by Puget that indicates that approximately 1,600 tons of large quarry rock was placed along the downstream edge of the dam in the 1930s. The presence of this material accounts for the very difficult driving experienced at location B. More details regarding the quarry rock are presented in a subsequent section entitled "Subsurface Conditions."

SEEPAGE TEST EXCAVATION

The purpose of the seepage test excavation was to evaluate seepage rates within the existing soils in the vicinity of the upstream right bank of the river and to evaluate stability of excavation side slopes. Information obtained from this seepage test is used in the design of the proposed earth cofferdams in this area (discussed later in this report). The location of the seepage test excavation is shown on the Site Plan, Figure 2.

The seepage test excavation, measuring approximately 30 feet in diameter and 14 feet in depth, is located immediately upstream of the dam on the right bank of the river. The excavation was completed by Deeny Construction of Seattle, Washington, using a 1 $^{1/4}$ cubic yard capacity track-mounted excavator on September 2 and 3, 1993. In general, the seepage test excavation was relatively easy to excavate.

Soil and Side Slope Conditions

Soil conditions encountered in the seepage test excavation consist of 3 to 4 feet of medium dense to dense silty sand with occasional gravel and scattered roots overlying well-graded, dense to very dense gravel with sand and cobbles. In the eastern (river side) half of the excavation, occasional debris (rubber tires, rotting logs etc.) was encountered in the soil from about 4 feet in depth to the bottom of the excavation. The excavation slopes varied from 1H:1V (horizontal to vertical) in the upper medium dense silty sand layer to near vertical in the lower dense sandy gravel layer. Ground water seepage typically was encountered at the top of the dense sandy gravel layer, which corresponds approximately to the static ground water level in the area.

Test Procedure

The dimensions of the seepage test excavation were determined by measuring the depths from the original ground surface to the ground surface along the sidewalls and base of the test excavation. This procedure was repeated twice along two sections, running north-south and east-west. Information obtained from the two cross section surveys were used to determine the volume of water in the seepage test excavation.

The depth to the water level in the test excavation was surveyed the morning after the excavation was completed (September 4). Standing water in the bottom of the excavation was approximately 4 feet deep at this time. This water was pumped out using a Gorman 16C2 diesel pump rated at 1,500 gpm (gallons per minute). The time required to pump the standing water out (referred to as the "drawdown" time) and the water level elevation at the completion of the pumping operation were recorded. The recharge rate, or the rate at which water seeps back into the test pit, was also recorded by measuring the water surface elevation in the test pit 1, 2, 3, 4, 5, 7, 10, 15, 20 and 30 minutes after stopping the pump. This procedure was repeated twice to obtain an average seepage rate.

Summary of Results

The drawdown and recharge rates for the soils in the vicinity of the seepage test excavation were calculated based on the volume of water pumped and the recharge time. In general, the drawdown rate ranged from about 0.8 fpm (feet per minute) for 3.8 feet of drawdown to 1.9 fpm for 1.3 feet of drawdown. This corresponds to a flow rate of 3.1 cfs (cubic feet per second) or 1400 gpm (gallons per minute) and 2.8 cfs (1300 gpm), respectively. The recharge rate averaged about 0.5 inch per minute for 1.25 feet of rise in water level. This corresponds to a flow rate of 0.06 cfs (25 gpm). Because of changing heads and water levels throughout these tests, it is not possible to determine soil permeabilities from the test results with any degree of accuracy. However, based on our experience with similar conditions, the measured drawdown rates indicate moderate soil permeabilities (i.e., between about 10^{-1} and 10^{-3} cm/sec (centimeters per second).

TIMBER SAMPLING AND CONCRETE CORING

General

The timber and concrete sections of the existing dam were sampled and cored to evaluate the existing condition of the dam. This information is used to evaluate the feasibility of reusing the existing dam section as a foundation for the replacement dam. Concrete cores were obtained from three locations in the downstream apron of the dam. The core locations, designated as C-1, C-2 and C-3, are shown in Figure 2. Core locations C-1 and C-2 which coincided with the locations of TW-1 and MW-1 were obtained prior to drilling. Core C-3 was obtained from the downstream key section of the existing dam. Timber samples were obtained prior to coring at all three core locations. GeoEngineers also examined the subgrade soils below the concrete to determine the firmness of these soils and to identify voids, if any, that may be present.

Sampling Procedure

Samples of wood from the timber overlying the downstream apron were obtained by cutting with a chain saw. The concrete core samples were obtained after removing portions of the overlying timber at the three core locations. The coring operations were completed by Seattle Coring Company of Kent, Washington. One 16-inch-diameter and one 6-inch-diameter core sample were obtained from location C-1. Two 12-inch-diameter core samples and one 6-inch-diameter core sample were obtained from location C-2. One 12-inch-diameter and one 6-inch-diameter core sample were obtained from location C-3. After the cores were removed, our field representative examined the subgrade soils directly beneath the dam, except at C-3 because this core did not extend to the subgrade soil. A 1/2-inch-diameter steel rod was used to evaluate the relative density of the soils and to identify voids, if any.

Results

The structural wood of the downstream apron timber several inches below the surface was generally in sound condition. However, the surface of several of the timbers was eroded by the flowing river water mixed with cobbles and debris.

Details of the compressive strength tests of the concrete core samples are presented in Appendix B. The results indicate a compressive strength ranging from about 3,500 to 4,500 psi (pounds per square inch). In general, the concrete cores were found to be in sound condition, with no obvious signs of deterioration. The aggregate appears to range from about 1/4 inch to 6 inches in size with a typical size on the order of 3 to 4 inches. Occasional minor voids were observed near the larger aggregate in one of the cores. Reinforcement steel was observed in the C-3 core, obtained from the downstream key section. No steel was observed in the other cores.

The dam foundation subgrade at core locations C-1 and C-2 was found to be firm and unyielding, with no discernable voids. Core C-3 did not penetrate to the bottom of the concrete. Subgrade soils below the concrete were also examined in the two borings previously drilled through the dam, B-1 and B-2 as part of our preliminary study. The subgrade soils at these two other locations were also identified as firm and unyielding, with no voids between the soil and bottom of concrete.

BORINGS AND LABORATORY SOIL TESTS

Two borings were drilled within the downstream apron of the dam for this final design study to provide additional subsurface information below the dam. The borings were completed as wells and are designated as TW-1 (pump test well) and MW-1 (monitoring well). Details of the well installation and subsequent pump test are presented in later sections of this report. The borings were drilled on September 9 and 10, 1993. The locations of these borings as well as previous explorations completed at the dam site are shown on the Site Plan, Figure 2. Logs of all borings drilled for the project, including MW-1 and TW-1, are presented in Appendix A. Details of the drilling methods and sampling techniques are also presented in Appendix A.

All soil samples were returned to our laboratory for further examination. Selected samples were tested to determine moisture content, dry density and grain size characteristics. A description of the tests performed and the results for this study as well as for previous studies are presented in Appendix B.

WELL CONSTRUCTION

Two wells, MW-1 and TW-1, were constructed below the dam by Holt Testing, Inc. on September 10, 1993. Wells TW-1 and MW-1 were constructed as the pumping and observation wells, respectively, for subsequent pump testing (described in the following section). The locations of TW-1 and MW-1 are shown in Figure 2.

TW-1 was constructed of 10-inch-diameter steel pipe with 0.060-inch slots between a depth of approximately 5.0 to 30.0 feet beneath the dam surface. A 10-inch-diameter blank steel pipe for TW-1 extended from the surface to a depth of 5.0 feet. To reduce water inflow at the surface, 12-inch-diameter steel casing was placed from the surface to a depth of 5.0 feet. The annulus between the 12- and 10-inch-diameter pipe was filled with bentonite to a depth of 4.0 feet below the dam surface. Native gravel soil surrounds the slotted pipe (no sand fill was used).

MW-1 was constructed of 2-inch-diameter PVC pipe with 0.02-inch slots between the depths of approximately 3.5 and 23.5 feet beneath the dam surface. MW-1 was completed with blank PVC pipe from the top of the slotted interval to the dam surface. Sand was placed around the slotted portion to act as a filter. Bentonite chips were used around the blank PVC pipe to seal the slotted portion from surface water. Well construction details for TW-1 and MW-1 are presented on the logs for these borings in Appendix A.

The well screen of TW-1 was developed by Holt using surge-block methods (a plunger that forces water in and out of the well) and by pumping water from the well prior to the start of the aquifer testing program. TW-1 was developed for approximately 4 hours on September 11, 1993.

GROUND WATER PUMPING TEST

General

A ground water pumping test program was conducted on September 13,1993 to determine aquifer characteristics at the well site and provide information regarding potential dewatering flow rates. The aquifer test program included three short-term constant rate tests and water level recovery tests. Holt provided the submersible pump, generator, piping and flow meter used to conduct the aquifer tests.

Test Procedures

Water levels in TW-1 and MW-1 were monitored while pumping from TW-1 at rates of approximately 200, 160 and 180 gpm (gallons per minute). Static water levels in TW-1 and MW-1 were 0.90 and 0.65 feet beneath the dam surface, respectively, prior to testing. Ground water levels were also measured during recovery periods after pumping was completed. In the 200 gpm test it took approximately 10 minutes to lower the water level in TW-1 to the pump intake depth. The pump was then turned off and water levels were allowed to recover before starting the 160 and 180 gpm tests. The 160 and 180 gpm tests lasted for three hours each, with recovery periods after each pumping period.

Results

Maximum drawdowns observed in MW-1 were typically less than 0.2 feet. Maximum drawdowns observed in the test well, TW-1, at the end of each constant rate pumping period are summarized in Table 1.

Table 1. Summary of Drawdown Measurements During Pump Tests

Measurement Location	Pumping Rate (ppm)	Duration of Pumping (hours)	Maximum Drawdown (feet)
TW-1	200	0.17	>18
TW-1	180	3	13
TW-1	160	3	6

Time-drawdown data from the three tests were used to estimate aquifer parameters at the site. The transmissivity and storativity of the aquifer were evaluated using the Theis and Cooper - Jacob nonequilibrium equations. Water level data were corrected by the Kruseman -DeRidder method to approximate unconfined aquifer conditions.

The calculated transmissivity of the aquifer ranged between approximately 2,800 and 5,200 square feet per day during the 160 gpm test and between about 200 and 700 square feet per day during the 200 gpm test. Aquifer storativity is estimated to be 8×10^{3} and 5×10^{-2} for the 160 and 200 gpm tests, respectively. Transmissivity and storativity values calculated from da ta obtained during the 180 gpm test were between those calculated for the 160 and 200 gpm tests. Based on an aquifer thickness of approximately 40 feet, the transmissivity values correspond to hydraulic conductivities between 2.4×10^{-2} and 6.0×10^{-3} cm/s (centimeters per second) (70 to 130 feet per day). This range corresponds well to permeability estimates derived from slug tests completed on the right bank during previous studies (Appendix B) and on correlations with grain size. By adjusting the pump rate to raise and lower the drawdown in the test well, we were able to delineate permeabilities (to some degree) between different intervals of soil. Specific permeability estimates corresponding to three approximate soil intervals are summarized in Table 2. It is cautioned that because of the nature of the pump test, these delineations are very approximate and actual permeabilities within each soil interval may be different from the values shown in Table 2.

Table 2. Permeability Estimates from Pump Tests

Pumping Rate gpm	Soil Interval Affected (feet)	Representative Soil Type	Estimated Range in Permeability (cm/s)
200	0-18	Mudflow	4.5 to 6.0×10^{-3}
180	0-13	Alluvial/Mudflow	1.3 to 2.5×10^{-2}
160	0-6	Alluvial	2.4 to 4.4×10^{-2}

SUBSURFACE SOIL CONDITIONS

GENERAL

Our understanding of soil conditions at the project site is based on our explorations (Appendix A), laboratory tests (Appendix B), pile driving tests (Appendix C), a geophysical study completed for the preliminary study (Appendix D), recently completed field test programs described above, a review of geologic data and historic information provided by Puget. It should be understood that in an alluvial and mudflow environment, soil conditions can change very abruptly from silt and sand to cobbles and boulders. Our description of soil conditions presented below is based on a reasonable interpretation of available data.

In general, the soils consist of two recent deposits, a sand and gravel alluvium deposit overlying a sand and gravel with silt mudflow deposit. The two soil deposits are similar from a foundation support standpoint and differ primarily in the amount of fines (percent passing the No. 200 sieve) and cobbles and boulders. The overlying alluvial deposits generally contain less fines and more cobbles and boulders than the underlying mudflow deposits. There are other minor differences in subsurface condition s, depending on location within the project site. We present a summary of soil conditions for each major component of the replacement dam project in the following sections. A summary of pertinent soil parameters for each soil layer is summarized in Table 3.

DAM ALIGNMENT

A soil profile along the dam axis interpreted from borings B-1, B-2, MW-1 and TW-2 and geophysical information is shown in Figure 6. The location of the profile is shown in Figure 2. The soils encountered in our explorations indicate the diversion dam is underlain by 7 to 10 feet (typical) of medium dense to dense sand and gravel with cobbles and occasional boulders (alluvium) over dense sand and gravel with silt and occasional cobbles (mudflow). Although we did not encounter boulders in our borings, occasional boulders should be expected in the mudflow deposits below the dam, based on our experience with this material. The fines content within the alluvial deposits encountered in our explorations typically ranges from 0 to 5 percent by dry weight. The fines content within the mudflow deposits typically ranges from 5 to 12 percent. No concrete debris or rubble was encountered in borings completed along the dam alignment. The soils below the dam appear to contain fewer cobbles and boulders than are present in the alluvial deposits upstream and downstream of the dam (discussed subsequently).

DOWNSTREAM OF THE DAM

Soil conditions downstream of the dam were interpreted from borings drilled through the dam, test pits completed in the river downstream of the dam and test pile driving at location B. We also reviewed information from Puget describing large quarry rock that was placed downstream of the dam in the 1930s. According to Puget accounting records, approximately 1600 tons of quarry rock "two yards or larger" in size was placed at the toe of the diversion dam between 1937 and 1939. This material apparently was placed to reduce scour in this area. Based on information from our field studies and from Puget, we have developed a subsurface cross section (perpendicular to the dam axis) in Figure 7. The location of the cross section is shown in Figure 2.

The surficial 7 to 12 feet of soil extending from the downstream edge of the dam to about 40 feet downstream consists of sand and gravel alluvial deposits mixed with large quarry rock (riprap) and concrete debris. A "ballpark" estimate of the areal limits of this fill/alluvial soil mix is shown in Figure 2. The delineation in the north-south direction is particularly vague due to lack of data. The riprap and concrete debris is very large (up to 6 feet in size). During the test pile program the excavator was not able to remove some of this material. The pile tips could not penetrate through the fill/alluvial mix, and refusal typically occurred 4 to 6 feet below the river bed. Test pit TP-10 and TP-11 was also excavated in this area to evaluate scour depths. Because the undisturbed native river bed soils are similar in nature to those deposited recently by the river, it is very difficult to delineate scour depth. Our best estimate is that scour may have extended as deep as 12 feet in the vicinity of TP-10.

Soil conditions encountered in test pits further than about 40 feet downstream of the dam consist of 7 to 12 feet of loose to medium dense sand and gravel with cobbles and boulders (alluvium) over dense sand and gravel with silt and occasional cobbles (mudflow). The size of the boulders in the alluvium typically ranges from one foot to 4 feet.

Table 3. Summary of Soil Properties

General Location	Depth Interval Below Ground Surface (feet)	Geologic Description	USC Class	(degrees)	C (psf)	Dry Density (pcf)	Fines Content (%)	Average Range in Permeability ($x10^{-2}$/sec)	Piping Potential
Along dam alignment	0-10	Alluvium	GP	40	0	135	0-5	2 to 5	Low
	Below 10	Mudflow	GP-GM	42	0	138	5-12	0.4 to 0.6	Low
Downstream of dam	1-12	Alluvium	GP	40	0	135	0-3	2 to 10	Low
	Below 12	Mudflow	GP-GM	42	0	138	5-12	0.4 to 0.6	Low
Upstream of dam	0-10	Alluvium	GP	40	0	135	0-5	2 to 5	Low
	Below 10	Mudflow	GP-GM	42	0	138	5-12	0.4 to 0.6	Low
Temporary diversion channel	0-7	Alluvium	GP	40	0	135	0-5	NA	NA
	Below 7	Mudflow	GP-GM	42	0	138	5-12		
Maintenance and control building areas	0-30 (approx.)	Mudflow	GP-GM	42	100	138	8-15	NA	NA
Right bank dike	0-10	Alluvium	GP	40	0	135	0-8	NA	NA

Note: Values presented above are based on limited subsurface information and therefore should be considered approximate.

UPSTREAM OF THE DAM

There is relatively little information regarding soil conditions in the river upstream of the dam, other than that described in a preceding section ("Surface Conditions") for the near-surface soils in the sand bars along the right river bank. No borings were accomplished in the river upstream of the dam. Our interpretation of soil conditions in the river channel upstream of the dam are based on the seepage test excavation, test pit TP-14 and test pile driving at location A. Our interpretation of soil conditions upstream of the dam is shown in Figure 7.

The soil conditions within the main channel of the river and upstream of the dam likely consist of 7 to 10 feet of medium dense to dense gravel with sand, cobbles and boulders (alluvium) over dense gravel with sand, silt and cobbles (mudflow). The soil conditions near the right bank consist of sand and gravel with cobbles and some silt. There are noticeably fewer large cobbles and boulders encountered in excavations near the right bank than in the main river channel.

TEMPORARY DIVERSION CHANNEL AREA

Soil conditions in this area are interpreted from test pit TP-15, boring B-4 and from the onshore pile driving test at location A (test A-3). At TP-15, the soils conditions consist of a very dense 3-foot-thick layer of sandy gravel with cobbles over medium dense to dense silty gravelly sand. The dense surficial soil appears to be desiccated mud flow deposits and the underlying medium dense soil appears to be mudflow deposits with a natural moisture content. Mudflow deposits harden when they dry out (desiccate). At boring B-4, the soil conditions consist of 7 feet of medium dense gravel with sand and cobbles (alluvium) over dense sand with gravel, silt and cobbles (mudflow).

MAINTENANCE AND CONTROL BUILDING AREA

Our interpretation of soil conditions in the proposed maintenance and control building area is based on our observations of the soil exposed along the left bank bluff and test pit TP-15 and boring B-4, which are located near the base of the bluff. No borings or test pits were accomplished above the base of the bluff. The soils in this area appear to consist of medium dense to dense gravelly sand with silt and occasional cobbles (desiccated mudflow) from the top of the bluff to the proposed footing elevation of the maintenance building (El. 675.5 feet).

RIGHT BANK DIKE

Test pits TP-1 through TP-6 were excavated along the newer section of dike in September 1990, prior to raising of the dike in 1992 (refer to "Surface Conditions"). Test pits TP-7 and TP-8 were excavated along the older (original) section of dike. All test pits encountered loose to medium dense gravelly sand with cobbles, boulders and varying amounts of silt to the depths explored. The dikes appear to have been constructed over the native sands and gravels (alluvium) using alluvial material. Therefore, it was difficult in some cases to distinguish between the dike material and the underlying native soils. The depths of dike fill range up to approximately 14 feet along the newer section and 3 to 6 feet along the older section. Although no test pits were excavated along the proposed right bank dike alignment (existing access road) from Station 0+00 to 8+00 (refer to Figure 1), we anticipate that soils in this area are similar to those described above for the older section of the dike.

HYDROGEOLOGIC CONDITIONS

Ground water in borings B-1, B-2, MW-1 and TW-1 was encountered initially at the downstream river level (about Elevation 659 feet). However, artesian conditions were encountered at a depth of approximately 30 feet (Elevation 631 feet) in boring B-1 when the air-rotary casing drilled into a clean sandy gravel zone. The water level in the casing rose to approximately 8 feet above the downstream river level (to about Elevation 667 feet, which is about 3 feet below the upstream water level). The artesian conditions continued for the duration of the drilling of boring B-1, which was completed the following day. An estimate of the flow rate from this boring was from 5 to 10 gpm. No artesian conditions were encountered in any of the other borings or wells.

The ground water level in boring B-4 was observed at a depth of 3 feet below the ground surface (at about Elevation 671 feet), which corresponds approximately to the upstream river level. The ground water level in the well installed at boring B-3 was recorded at approximately 12 feet below the ground surface (Elevation 661 feet), which is approximately 10 feet below the level of the river upstream of the diversion dam.

No ground water seepage was encountered in the onshore test pits. However, slight to moderate caving was encountered in test pits TP-4 and TP-5 at the east end of the right bank. Moderate seepage was observed below about 5 feet in the seepage test excavation located on the right bank, just upstream of the dam.

The ground water levels observed in our explorations indicate a relatively complex ground water system. Along the right bank near the dam, boring B-3 showed ground water at Elevation 661 feet or about 10 feet below the level of the river located about 25 feet away. Wells installed in other explorations completed by GeoEngineers for the hatchery showed ground water elevations between 660 and 665 feet at a distance of 300 to 600 feet from the right bank of the river (Sverdrup Corporation, 1967; refer to List of Reference Reports at end of text). This suggests that the regional ground water level on the right bank is 7 to 10 feet lower than the river level upstream of the diversion dam. No water seeps were observed along the right bank. We interpret this information to imply that water from the river is discharging into the aquifer below the right bank.

Seeps and wet zones of soil were observed along the left bank of the White River upstream and downstream of the diversion dam, The ground water level encountered in boring B -4 and test pit TP-15, located about 15 feet from the river, is about 3 feet above the river level. This indicates ground water flow is toward the river.

protection without significantly reducing existing dike stability. The existing access road also appears to be suitable for placement of additional fill for new dike construction.

Based on site reconnaissance and a review of aerial photographs, it appears that relatively little erosion has occurred along the right and left banks of the White River within 1/2 mile upstream of the dam since 1936 (reference 1936 aerial photograph). Existing riprap protection of the right bank dike has adequately protected the dike fill with minor repair every few years.

Detailed conclusions and recommendations regarding temporary diversion and dewatering, reuse of the existing dam structure, foundation support, right bank dike construction, river channel and dike erosion protection and seepage considerations are presented in the following sections.

TEMPORARY DIVERSION AND DEWATERING

Cofferdams

We recommend that all cofferdams in the river be constructed using sand and gravel, protected by riprap, as needed. The specifications for cofferdam construction depend on location within the river. In relatively low velocity areas, such as along the right bank, sandier soil with minimal protection will be feasible. In higher velocity areas, such as upstream in the main river channel, more significant protection of the cofferdam will be required. We have developed schematic diagrams showing recommendations for three cofferdam designs which differ primarily between the level of riprap protection. Type I is for use in the higher energy areas; Type II is for use in moderate energy areas; and Type III is for use in low energy areas. The three recommended designs are shown in Figure 8. We anticipate that Type I will be used in the higher flow areas in the main river channel in both Phase I and Phase II construction. Type II will likely be used in the lower flow areas downstream of the Type I cofferdam in the Phase II construction and to the right of the Type I cofferdam in the Phase I construction. Type III will likely only be used between the right bank and the Type II cofferdam in Phase I construction. During normal river flow, slope protection of the cofferdam near the right bank where velocities are very low is probably not needed. However, we recommend that the minimal riprap protection shown for Type III in Figure 8 be placed over the slope of the cofferdam for safety precautions, in case the river flow unexpectedly increases during construction.

ZONES NEED TO BE LOCATED ON PLAN SHOWING COFFERS. THIS IS UNCLEAR FIG. 3

All cofferdam types include three basic components: a sand and gravel core, riprap protection and a geomembrane barrier between the sand and gravel and riprap to reduce water flow through the cofferdam. A minimum crest width of 12 feet is recommended to provide adequate access for construction vehicles (including a drill rig to install dewatering wells). Design parameters and construction considerations for each are discussed further below.

We understand from Puget that it is probable that river levels (and therefore, velocity) will be lowered during construction of the cofferdams in the main river channel. If the river velocity is sufficiently low, it is possible that the sand and gravel used as the core material will remain essentially in place until the riprap protection is constructed. However, some movement of the sand and gravel should be expected. It may be necessary, if the river velocity is too high and protection without significantly reducing existing dike stability. The existing access road also appears to be suitable for placement of additional fill for new dike construction.

Based on site reconnaissance and a review of aerial photographs, it appears that relatively little erosion has occurred along the right and left banks of the White River within 1/2 mile upstream of the dam since 1936 (reference 1936 aerial photo graph). Existing riprap protection of the right bank dike has adequately protected the dike fill with minor repair every few years.

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We understand from Puget that it is probable that river levels (and therefore, velocity) will be lowered during construction of the cofferdams in the main river channel. If the river velocity is sufficiently low, it is possible that the sand and gravel used as the core material will remain essentially in place until the riprap protection is constructed. However, some movement of the sand and gravel should be expected. It may be necessary, if the river velocity is too high and movement of the sand and gravel is too great, to first place a riprap barrier upstream of the sand and gravel core location to reduce water velocities. This technique is intended to allow the sand and gravel fill to remain in place until the upstream slope can be covered with riprap protection. Other methods of cofferdam construction may be also feasible. We recommend that specific techniques for cofferdam construction be made the responsibility of the contractor who has experience with this type of construction.

Riprap. We recommend that the riprap consist of well-graded, durable rock. We recommend that the riprap be tested for slaking and durability prior to placement. A sample of the riprap should be submitted to the Engineer at least two weeks before cofferdam construction begins.

Sand and Gravel Core. The core material should consist of well-graded sand and gravel with less than 5 percent fines by dry weight. The gravel content should be at least 40 percent by dry weight. The alluvial deposits in the upper 10 feet of the river bed typically meet this specification, based on our explorations. We anticipate that a cost-effective method to construct the cofferdams is to use the upper alluvial material from the excavations in the river or from upstream sand and gravel bars to construct the sand and gravel core. Other off-site material sources meeting the above specifications would also be appropriate.

Geomembrane. The geomembrane shown in Figure 8 in between the riprap and sand and gravel core is used to reduce water flow through the earth cofferdam. This membrane must be sufficiently durable to withstand the punching and tearing stresses during placement of the riprap and yet be flexible enough to place in difficult conditions (in the water). It must also possess a relatively low permeability. Based on our experience with similar types of projects, we anticipate that 30-mil PVC or Hypalon membranes may be suitable. Other types of membranes may also be suitable. We recommend that the contractor be required to submit specifications for the proposed geomembrane for review prior to construction of the cofferdams.

Construction Dewatering

The results of our tests and our analyses indicate that water seepage below a typical earth cofferdam into a 10-foot-deep excavation may be on the order of 3 to 6 gpm per unit length of cofferdam in the main river channel of the dam site. We anticipate that the lower part of the range may be more appropriate for Phase I construction, particularly near the right bank where excavations may be in slightly less permeable soils and where excavations are not expected to extend much deeper than the existing dam foundation. Excavations for Phase II construction will be deeper and will probably extend into more permeable soils. Therefore, we expect that water seepage into these excavations will be in the upper portions of the estimated seepage range.

This estimate for water seepage is based on the earth cofferdam configurations shown in Figure 8 and on ranges in permeabilities presented in the "Subsurface Conditions" section of this report. Most of this water inflow will likely be through the upper 10 feet of moderately permeable alluvial deposits, since the geomembrane in the cofferdam will significantly reduce water flow through the cofferdam.

The alluvial soils in the river can vary over short distances. It is possible for localized zones of highly permeable soils such as gravel with abundant cobbles to be present. In these areas, significantly higher flow rates should be expected. Inflow rates on the order of 20 to 40 gpm per unit length of cofferdam could occur in these localized areas.

Based on the estimated inflow rates presented above, it is our opinion that dewatering wells will likely be required in most of the areas away from the right bank to sufficiently lower the ground water level and reduce water inflow in the excavations. The rate of ground water removal in the wells will be higher than the estimates for seepage rates presented above. The dewatering system should be designed to lower the water in the excavation a minimum of 3 feet below the base of the excavation. We recommend that the contractor be made responsible for designing the dewatering system (including estimating dewatering rates in the wells).

It may be possible to use sumps and pumps near the right bank or other areas where less permeable, finer-grained soil is present and where planned excavations are relatively shallow. We were able to adequately dewater the moderate-sized seepage test excavation located near the right bank using a large diesel-powered pump. However, because of the large size of the excavation and the presence of more permeable soils in the main river channel, we do not expect that sumps and pumps alone will be adequate for this area.

Properly designed and installed dewatering wells should adequately reduce water inflow as well as minimize boiling and softening of the excavation subgrade soils. It will be important to maintain stable conditions along the base of the excavation and to prevent loosening of the soils that will increase post-construction settlements.

We recommend that GeoEngineers be retained to review the design to verify that the contractor has properly interpreted our data and recommendations. As part of the design, the contractor should indicate how unanticipated changes in water inflow rates will be addressed during construction. For example, installing an additional well in between two other wells may be appropriate to control a localized zone of high water inflow. The contractor may also need to supplement the wells with sumps and pumps to control water inside the excavation, particularly in the radial gate area where localize excavations may be up to 15 feet deep.

Temporary Excavations In the River

Temporary excavations will be required inside the earth cofferdams to allow for construction of the replacement dam. We recommend that appropriate dewatering systems (discussed above) be in place and operating prior to beginning excavation operations. This will allow excavations to proceed with a higher degree of stability.

We anticipate that the excavations can be accomplished by large backhoes or excavators. Our experience with excavations in the river suggests that an excavator equipped with a large bucket can dig effectively in the upper alluvial gravel, cobbles and boulders. However, the excavator did encounter difficult digging downstream where the riprapand concrete debris are present. We also anticipate difficult digging in the lower dense mudflow deposits.

Based on our explorations, it appears that the soils in the river are predominantly gravel with sand, cobbles and boulders. We recommend that temporary slopes in this material be designed at 2H:1V. It may be feasible to steepen the slopes in some areas during construction if soil and seepage conditions permit. GeoEngineers should be consulted prior to constructing any temporary slope steeper than 2H:1V.

Temporary Diversion Channel

It will be necessary to use sheet piles for the temporary diversion channel in order to properly contain the high design flows. Based on our explorations, we anticipate very difficult driving in the upper 5 to 8 feet of the ground surface. Prior to installing sheet piles, it will likely be necessary to excavate the upper dense mudflow, cobbles and boulders. We anticipate that driving below this level will also be difficult because of the presence of cobbles and boulders. We recommend that the contractor be prepared to predrill or prespud the soils along the proposed alignment. In addition, it may be necessary to remove large boulders that may be present below the sheet pile tip(s) during driving operations. Heavy section sheet piles equipped with pile tips are recommended. A large vibratory hammer will likely be required to achieve adequate penetration.

We understand that structural braces will be required across the top of the channel to support an access way. Because of this bracing, we anticipate that the sheet piles will not need to be driven very deep into the ground. However, there still may be areas where the sheets cannot be driven sufficiently deep below ground surface, even with the techniques suggested above. Additional, localized bracing may be needed in these areas.

EXISTING DAM STRUCTURE

It is our opinion that the existing dam structure may be reused as the foundation for the fixed panel and rubber weir portions of the replacement dam. Specific conclusions and recommendations regarding reuse of the existing dam are presented below.

The structural wood of the timber overlying the concrete apron is sound and suitable for foundation support. However, many of the surficial timbers have been severely eroded in places due to the flowing water and the action of cobbles and boulders being transported over the dam. We therefore recommend that the upper exposed layer of the timber on the downstream apron be removed prior to construction of the new dam structure.

The concrete in the downstream apron appears to be sufficiently sound for foundation support of the new dam structure. The strength of the concrete is above 3,000 psi and no significant voids were observed. It may be appropriate to remove the surficial weather concrete surface prior to construction of new dam components.

No voids were observed between the bottom of the existing concrete and the subgrade soils. The subgrade soils were observed to be firm and unyielding. Based on the available information, we recommend using the allowable soil bearing pressures versus estimated settlement relationships for the design of the replacement dam in this area, presented in Table 4:

Table 4. Soil Bearing Pressures for the Existing Dam

Allowable Soil Bearing Pressure (psf)	Estimated Settlement (inch)
1,000	0.2
2,000	0.3
3,000	0.5
4,000	0.7
5,000	0.8
6,000	0.9

The settlement estimates presented above are based on the following assumptions:

- the width of the mat foundation is the same as the existing dam width.
- the base of the foundation will remain at the present elevation.
- the soil conditions at the base of the existing foundation consist of medium dense to dense sand and gravel alluvium.

We recommend limiting bearing pressures to 6,000 psf or less for structures constructed over the existing dam. Most of the settlements presented above should occur as the new loads are applied due to the granular nature of the soil. We estimate that differential settlement across the width of the foundation may be on the order of 1/2 or less of the total settlement, depending on the specific loading conditions and variability of soil conditions. Post-construction settlement is expected to be negligible.

FOUNDATION SUPPORT

Radial Gate Section

The radial gate section of the replacement dam may be supported directly on undisturbed native soils. The soils at the proposed bottom of foundation level (El. 652 feet, typical) are expected to consist of either mudflow deposits or a thin layer of alluvium over mudflow deposits. Both types of deposits consist of dense sandy gravel with cobbles and relatively minor amounts of fines and are suitable for support of the concrete mat foundation for the radial gates.

We anticipate that some loosening of the soils at the base of the excavation may occur as A result of disturbance from the excavator. The base of the excavation should be compacted with a vibratory roller to a firm and unyielding state after the design elevation has been achieved. We recommend that the base then be evaluated by proofrolling to identify any localized loose areas. Any loose or disturbed areas that cannot be adequately compacted in place should be removed and replaced with crushed rock compacted to a firm and unyielding condition.

Settlement estimates in granular soil are primarily dependent on the elastic properties of the soil and the type of analysis. The elastic soil properties can vary widely, depending on density, gradation and other factors. We have completed settlemen t analyses using several formulae and estimated soil properties based on published data. Table 5 shows a relationship between allowable soil bearing pressure below the proposed 5 -foot-thick concrete mat for the radial gate section and estimated settlement based on average soil parameters.

Table 5. Soil Bearing Pressures for the Radial Gate Section

Allowable Soil Bearing Pressure (psf)	Estimated Settlement (inch)
2,000	0.2
4,000	0.4
6,000	0.7
8,000	1.0

The settlement estimates presented above are based on the following assumptions:

- the radial gate mat is 5 feet thick and approximately 40 by 65 feet in plan.
- the bottom of the main portion of the mat will be at Elevation 652 feet.
- soil conditions at the base of the mat consist of dense sand and gravel alluvium or mudflow deposits.

Because of the granular nature of the soil, settlements will occur essentially as the pressures are applied. Therefore, post-construction settlements at the radial gate section should be estimated by using only the net increase in foundation pressure that is expected from operation of the gates. Settlement resulting from dead weight of the structure will have already occurred prior to operation of the gates. Differential settlements below the mat will depend primarily on the stiffness of the mat, the distribution of the loading on the mat and the soil conditions below the mat. In general, we anticipate that differential settlements below the planned 5-foot-thick mat may be on the order of one quarter of the total settlement.

We recommend that the differential settlement between the radial gate mat and the existing dam foundation be considered in the design of the joint between the two sections. Differential settlements between these two areas could be as much as the total settlement estimates presented above. Elastic differential settlements will occur between these two sections for any type of foundation, including piles, because of the different loading conditions.

Maintenance and Control Buildings

Soil conditions at the proposed maintenance and control buildings are expected to consist of dense mudflow deposits. This material should provide suitable support for shallow foundations provided the soil at the footing level is undisturbed. The footing subgrade should be evaluated to identify any areas of loose or disturbed soil. All areas of loose soil should be recompacted in place to a firm and unyielding condition or should be replaced with compacted sand and gravel.

Provided the subgrade is prepared as described above, we recommend that an allowable soil bearing pressure of 3,000 psf be used. Minimum footing sizes for individual spread footings should be 3 feet. Strip footings should have a minimum width of 18 inches. All exterior footings should be buried at least 18 inches below final grade.

Settlements of footings loaded as recommended are expected to be less than about 1/2 inch. This settlement will occur essentially as the loads are applied.

Upstream Apron

The upstream apron will consist of a 2-foot-thick, 100-foot-long concrete slab located upstream of the radial gate section as shown in Figure 3. The foundation loads are expected to be relatively light. The bottom elevation of the slab will range from 655 feet near the dam to 657 feet upstream. Soil conditions at these elevations are expected to consist of medium dense to dense alluvium. We recommend that an allowable soil bearing value of 2000 psf be used for the upstream slab. Post-construction settlement at this load is expected be less han 1/4 inch.

Fish Access Structures

New fish access structures are planned near the right and left sides of the dam. These structures typically consist of concrete mat slab supporting concrete walls. Foundation loads are expected to be moderate. The base elevations of the structures vary at each location. In general, we anticipate that the soil conditions will consist of medium dense to dense alluvium. An allowable soil bearing pressure of 2,000 psf may be used for the foundation design of these structures. Post-construction settlement is expected to be less than about 1/4 inch for this design pressure.

Permanent Excavations

A significant cut will be required in the maintenance building area to achieve the design grade. This area is expected to be underlain primarily by dense mudflow deposits. We recommend that the permanent cut slope in this area be no steeper than 11/2H:1V. We also recommend that the slope be vegetated as soon as practical after the excavation is completed to reduce erosion, If the cut is made during a wet period, it may be necessary to hydroseed the slope. Surface drainage above the slope should not be allowed to run over the top of the slope and should be directed to a collection area and tightlined to an appropriate discharge point.

RIVER CHANNEL AND DIKE EROSION PROTECTION

Right Bank

The river banks and the right bank dike located within a half mile upstream of the dam appear to be fairly stable at this time. The newer portion of the dike (Station 13+00 to 25+00) has had a layer of riprap added to the slopes in 1992 as part of dike reconstruction in this area. The older portion of the right bank dike that will still be used (Station 0+00 to about 13+00) appears to be stable with little to no evidence of erosion. Vegetation is well-established on the dike slopes and most of the shoreline in this area is somewhat protected by the sand and gravel bars on this side of the river. Therefore, it is our opinion that additional erosion protection may not be warranted in the older portion of the north dike. It may be appropriate to fill in areas with riprap where significant voids exist between large rocks or bare ground is present. We consider placement of riprap in these areas optional since erosion of the right river bank in these areas appears to be negligible.

No explorations have been completed in or near the access road. Based on evaluation of surface soils and extrapolation of the closest explorations, we anticipate that the road is comprised primarily of sand and gravel with relatively minor percentages of silt. The river side of the 2- to 4-foot-high road is protected by 12-inch-minus quarry spalls. The existing access road appears to be inherently stable and suitable for placement of additional fill, if needed. However, we recommend that the near-surface soils of the access road be well compacted using a vibratory roller prior to placing any fill. Although unlikely, it is possible that some areas along the toe of the access road may be underlain by soft, native soil deposits. Any soft or loose areas, if present, should be removed and replaced with compacted structural fill. A determination of the suitability of subgrade soils should be made at the time of construction.

The height of the access road may be increased by placing suitable fill over adequately prepared subgrade. We recommend that the fill consist of sand and gravel containing less than about 12 percent fines (percent passing the number 200 sieve, by dry weight). The fill should be compacted in lifts not exceeding 12 inches loose thickness to a minimum density of 90 percent of the maximum dry density as per ASTM D-1557. We anticipate the fill may be obtained from various on-site and off-site sources. Dredge soil from the nearby Woslagel Basin pit is available as fill. Based on our preliminary evaluation of these materials, it is our opinion that may be suitable for access road construction. Additional riprap may be needed along the river side of the access road, depending on the adequacy of the existing riprap and the material used in heightening the road.

Left Bank

Some minor erosion and bank recession appear to be present along the left bank bluff just upstream of the dam, Erosion of this bluff has occurred relatively slowly historically based on the review of the aerial photographs. It is not possible to accurately estimate changes in historical erosion rates from aerial photographs due to lack of resolution. The average rate of bank erosion appears to have been about 0.1 to 0.2 feet per year over the last 50 years. It is likely that bank erosion will continue at this rate, provided there are no changes in river flow or geometry. We anticipate that bank erosion will be intermittent with slabs of soil falling into the river during high flow rates and little to no erosion at other times. The rate of bank erosion may increase if upstream river conditions (including flow rates or river geometry) change.

It is possible that a slight increase in bank erosion may occur over historical rates due to a narrower channel and a slightly sharper bend that has developed in this part of the river over the last 20 years. However, we do not expect any rate increase to be significant. Based on our findings, we do not anticipate that bank erosion protection will be needed along the left bank bluff if historical erosion rates, or slightly higher, are acceptable. Should the rate of bank erosion increase in the future, the need for bank erosion protection should be evaluated.

We recommend that a monitoring system be installed at the top of the bluff in this area to obtain more information on the erosion rate. The monitoring system may consist of steel st akes driven into the ground and surveyed, horizontally and vertically. The monitoring frequency should be at least annually for the first few years and, depending on the results, may be reduced subsequently.

If the historical erosion rates are judged to be unacceptable and additional river bank protection is deemed appropriate, it may be accomplished by several methods. These include flattening the bluff and protecting the slope with suitable riprap material or constructing training groins. Slope flattening may be accomplished by either placing fill along the toe or by cutting back from the top of the bluff. Training groins, if used, will require placement of large rock (i.e., 2 to 3 feet in size) in the river in a configuration such that sediment will natur ally be deposited behind the groins. We anticipate that the upstream tie -in point to the riverbank will be approximately 1,000 to 1,200 feet upstream of the diversion dam. Details of the groin configuration may be developed as needed.

The left bank upstream of the bluff is covered with well-established vegetation. Erosion protection of the left bank beyond about 1,000 feet upstream of the dam does not appear to be warranted.

Downstream

Significant erosion and subsequent deposition of river bed deposits has occurred downstream of the dam. The primary location of this erosion is within the main river channel (starting from the left end of the dam to about 150 north (right) of the left end). Based on hydraulic model studies completed by Northwest Hydraulic Cons ultants, Ltd. (1992), it appears that the significant erosion after the new dam is constructed will occur in the same area. Specifically, Northwest Hydraulics Consultants, Ltd. predicts that significant erosion will occur within about 100 feet downstream of the two new radial gate sections (RG1 and RG2) and left rubber weir (RW1). Their studies indicate that scour may occur as deep as 20 feet or more, assuming no riprap is present. As previously discussed, approximately 1,600 tons of riprap had been placed in this area in the 1930s. We encountered some of this material, as well as large concrete debris, in several of our test pits. This material appears to have reduced downstream erosion of riverbed material.

However, it may not be sufficient to control erosion under hydraulic conditions that will be imposed by the new dam configuration and operation. It may be appropriate to place additional large (4 feet and larger) riprap in this area to reduce post-construction erosion.

SEEPAGE CONSIDERATIONS

Seepage below the Existing Dam

GeoEngineers completed seepage analyses below and around the existing diversion dam to develop baseline seepage rate estimates for use by the design team during the preliminary design of the project. We used a standard flow net analysis method to estimate the seepage below the existing dam. The estimates of seepage below the dam presented in our May 4 preliminary report range from 1 to 10 cfs (cubic feet per second). This estimate is based on permeability values estimated from slug tests on the right bank and grain size relationships. The permeability estimates derived from the recently completed pump test indicate that the soil permeability below the dam is lower than previously estimated. Based on updated permeability data, we estimate that the range in seepage under the existing diversion dam to be about 1/2 to 2 cfs. This corresponds to 0.02 to 0.1 percent of the maximum intake (2,000 cfs) into the flume. These calculations assume 11 feet of differential head across the dam. Since seepage rate is directly proportional to differential head, seepage rates at other heads may be estimated through linear extrapolation of these values.

Piping Potential

Based on grain size distributions of the soils below the dam and our experience with the local river deposits, it is our opinion that the potential for piping below or around the dam is relatively low. The soil consists of a moderately graded mix of gravel and sand with cobbles and small amounts of silt. We understand that HCR plans to install drains through the existing dam to reduce uplift pressures. We recommend that the bottom of the wells be properly screened to prevent transport of the finer soils up through the wells.

Downstream End Walls

We understand that end walls will be used to reduce scour along and under the downstream side of the replacement dam. We recommend that the walls extend to the mudflow deposits which were typically encountered in our explorations within a depth of 10 feet of the surface of the downstream apron. Because of the presence of large boulders and riprap in this area, sheet piles are not considered feasible. Several methods of construction of an end wall have been considered, including a cast-in-place concrete wall about 10 feet downstream of the existing end wall, a cast-in-place concrete wall adjacent to the existing end wall, and a tangent concrete pile wall constructed adjacent to the existing end wall. The cast-in-place concrete walls will require excavations close to the existing end wall and may result in caving of soil from beneath the existing end wall and dam foundation. If this occurred, it would affect the integrity of the foundation. Therefore, we recommend that the end wall be constructed using the tangent pile (or similar) method.

A tangent concrete pile wall consists of a row of concrete piles (each pile is typically 1 to 3 feet in diameter) such that the sides of each pile are touching each other. Thus, the row of individual piles acts as a wall. Because of interference between adjacent piles, the wall must be constructed in stages. The wall is constructed by drilling holes at about 3 pile dia meters apart and filling the holes with concrete and appropriate reinforcing steel. After the concrete in these holes has set, the contractor drills holes in between the first set of piles and fills these holes with concrete. Finally, the voids between the first two passes of hardened piles are drilled and concrete poured. After the wall is completed, it is tied structurally at the top to the existing (or replacement) dam foundation using grouted bolts or other suitable connections. This provides added lateral support. Existing riprap or other debris may be encountered during drilling. It may be necessary to prespud or drill smaller holes (e.g., using an air track drill rig) in these areas, if encountered, to break up the obstruction.

LATERAL EARTH PRESSURES

Active earth pressures on the upstream dike walls or downstream end walls may be evaluated using a triangular-shaped equivalent fluid earth pressure of 18 pcf (pounds per cubic foot) times the height of soil behind the dam wall or cutoff. Hydrostatic and dynamic water pressure should be added to the equivalent fluid earth pressure.

Resistance to lateral loads may be developed through friction between the foundation base and the underlying soils and by passive earth pressure along buried foundation components. Friction along the base of the foundation may be computed using a coefficient of friction of 0.8 applied to vertical dead-load forces. This includes no factor of safety. We recommend using a factor of safety of 1.5 to 2.

In addition to base friction, passive pressure along the downstream dam wall or cutoff may also be used to resist lateral loads. An equivalent fluid earth pressure of 500 pcf times the height of soil is considered appropriate for the passive case under submerged conditions. This value includes no factor of safety. We recommend using a factor of safety of at least 1.5. The depth of soil in the passive zone may need to be reduced if there is a potential for scour in this zone.

LIMITATIONS

We have prepared this report for use by Puget Sound Power & Light Company and HDR Engineering, Inc. in design of a portion of this project. The data and report are based on interpretation of available subsurface information. Our conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty or other conditions, express or implied, should be understood.

We trust this information meets your needs. If you have any questions regarding this information, please contact us.

Respectfully submitted,

GeoEngineers, Inc.

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Associate

Gordon M. Denby, P.E.
Principal

DWM:GMDnrw
Document ID: 0186227.MI6

Three draft copies submitted

LIST OF REFERENCE REPORTS
WHITE RIVER PROJECT

GeoEngineers, Inc. "Technical Memorandum No. 13. Preliminary Design Report, Geotechnical Engineering Services, Proposed Diversion Dam Replacement, White River Hydroelectric Project, Buckley, Washington," GEI File No. 0186-115-R06, May 4, 1992.

GeoEngineers, Inc. "Preliminary Report. Headwater Benefits, White River Basin," June 3,1983.

GeoEngineers, Inc. "Report. Reconnaissance for White River Flume," July 16, 1984.

GeoEngineers, Inc. "Report. Preliminary Osceola Mudflow Exploration for Prop osed Powerhouse at the White River Lined Canal," March 21, 1985.

GeoEngineers, Inc. "Report of Geotechnical Consultation, Preliminary Embankment Stability Basins, White River Sediment Basins," June 6, 1985.

GeoEngineers, Inc. "Report. Geotechnical Investig ation, Proposed Transformer Addition, White River Plant," February 14, 1986.

GeoEngineers, Inc. "Progress Report No. 1. Design Memorandum, Geotechnical Services, Tailrace Elements, White River Power Project," June 24, 1986.

GeoEngineers, Inc. "Report of Ge otechnical Services, Subsurface Contamination Study, White River Headworks," February 27, 1987.

GeoEngineers, Inc. "Report, Geotechnical Studies, Phases I & II, White River Flume Rebuild, Headworks Section," August 31, 1987.

Sverdrup Corporation "Draft Report. Detailed Hatchery Siting Study On or Near the White River," June 1987.

Ott Water Engineers, Inc. "White River. Facility Siting Study," August 1986.

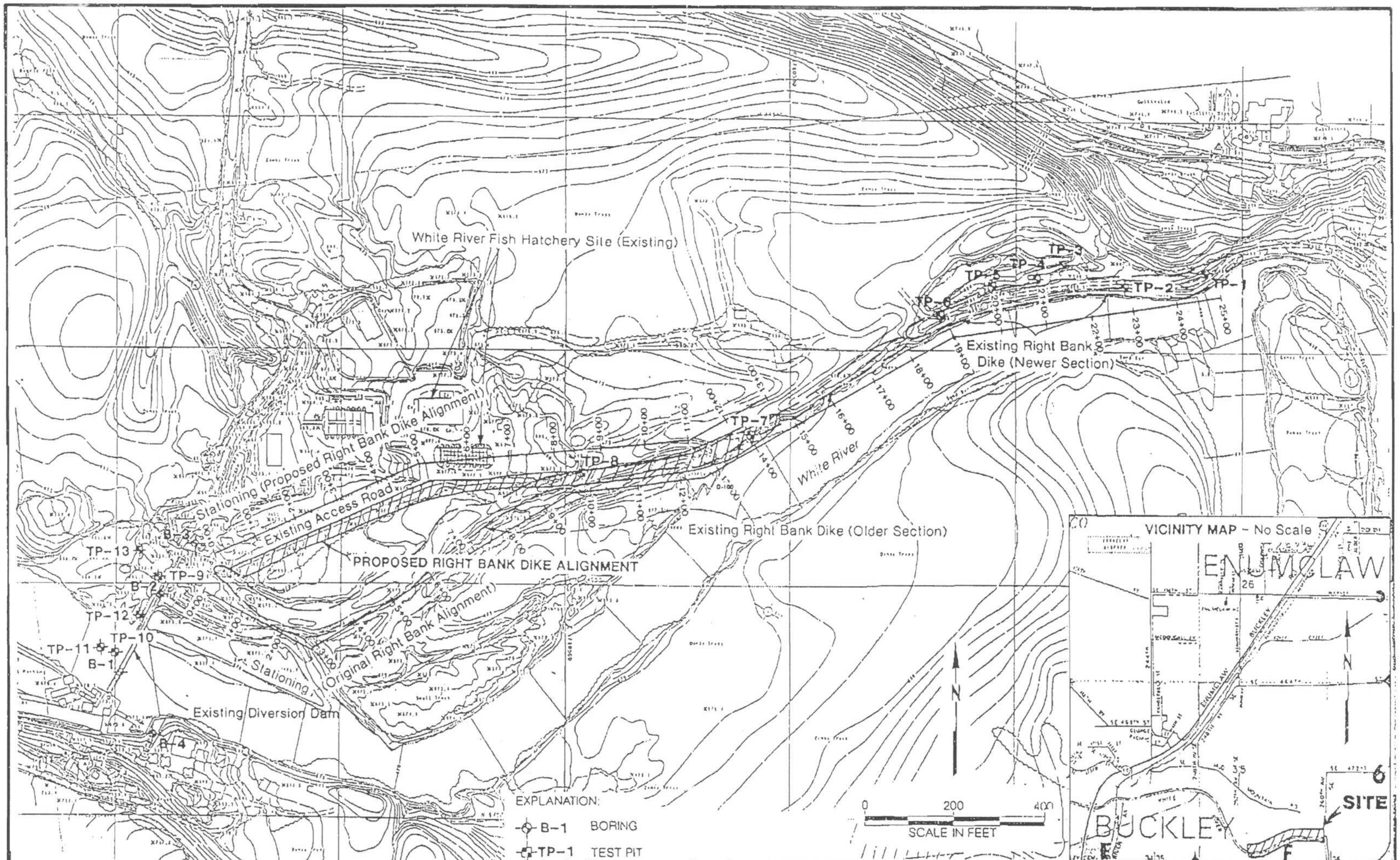
Ebasco Services, Inc. "Lined Canal Replacement for White River Project," January 1989.

Puget Power "Application for License, Major Project at Existing Dam, White River Project," November 1983.

Dunne, Thomas "Sediment Transport and Sedimentation Between RM's 5 and 30 Along the White River, Washington," August 1986.

Ebasco Services Inc. "White River Project Sediment Studies," July 1988.

Ebasco Services Inc. "Seismic Refraction Survey," November 1982.



Reference: Photogrammetric map entitled 'White River,' by Walker and Associates, Inc., dated 12/11/89.

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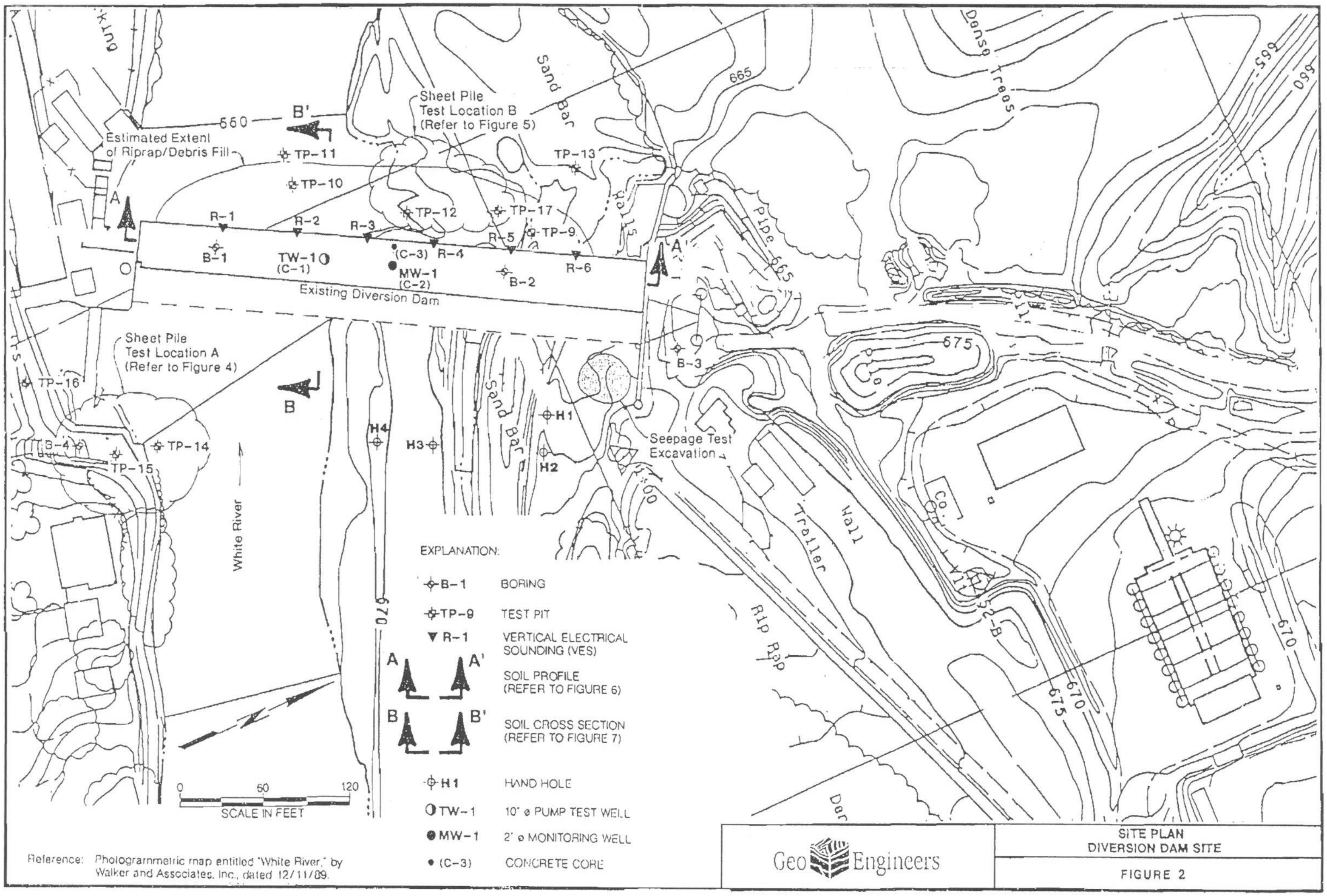
VICINITY MAP AND SITE PLAN

FIGURE 1



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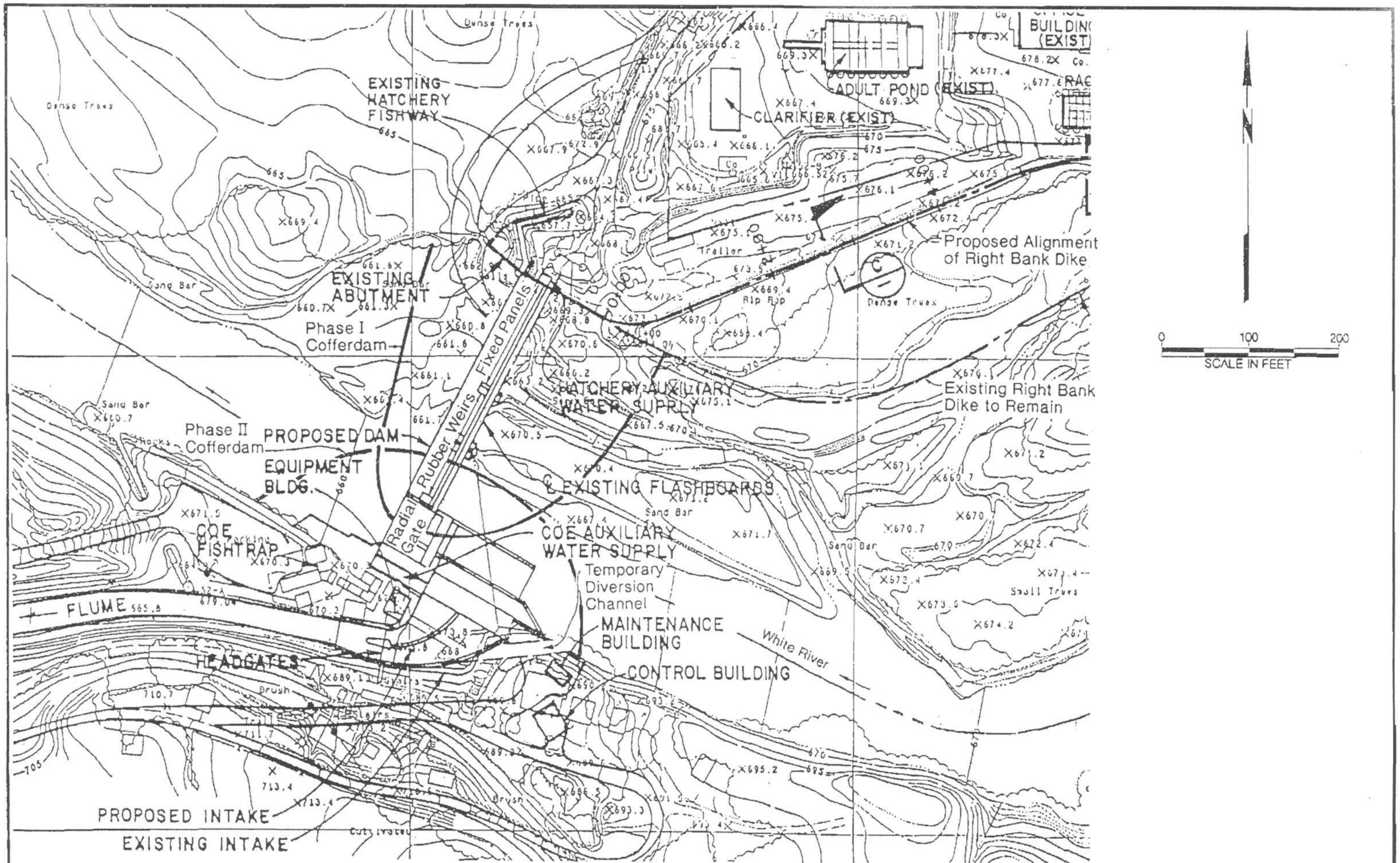

Reference: Drawing entitled "Project Site Plan, Drawing No. 3," by HDR Engineering, Inc., dated 05/21/92

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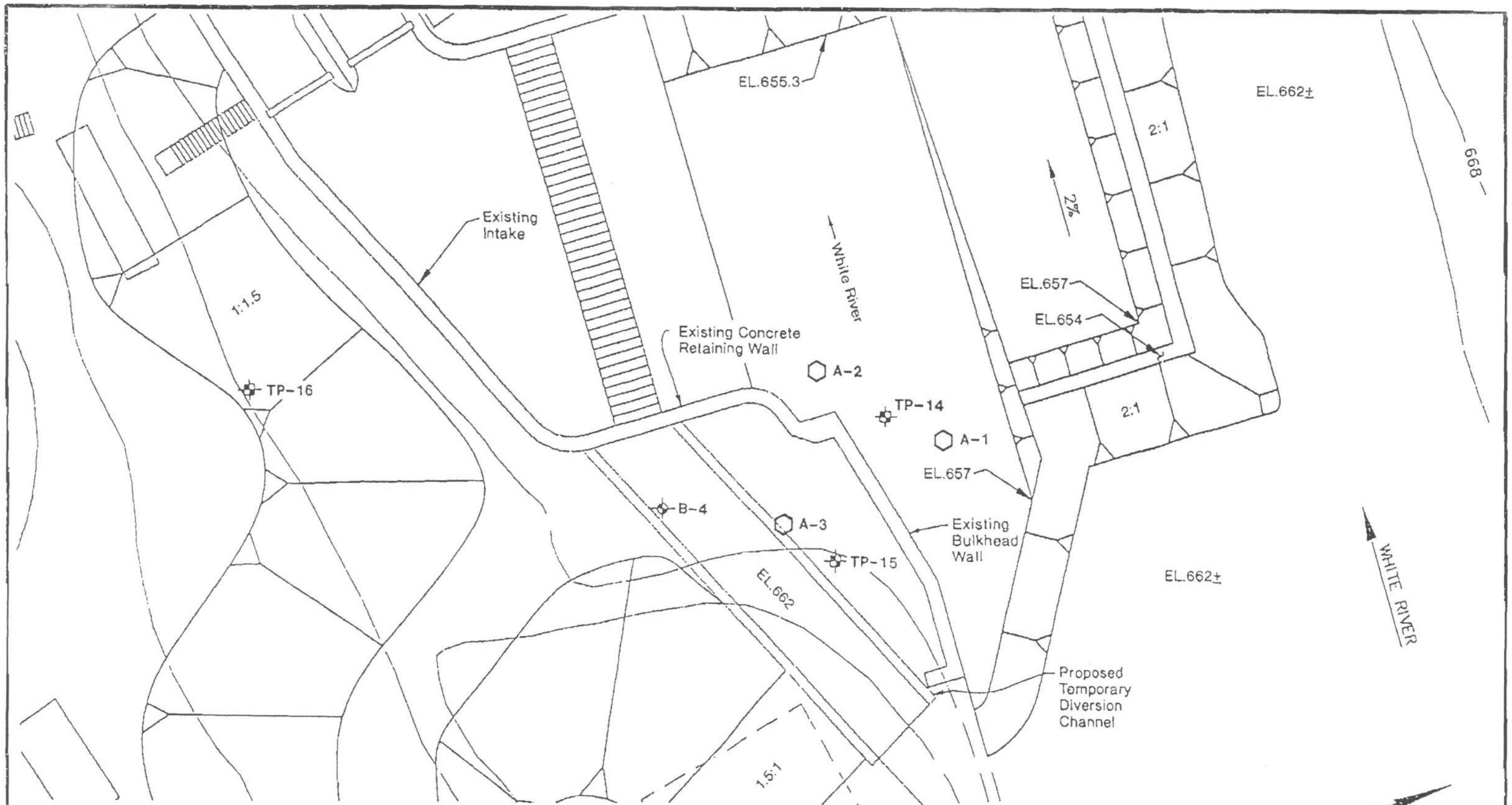
PROPOSED DIVERSION DAM COMPONENTS

FIGURE 3

0186 127 R06 DAM BDY HH:HS



EXPLANATION:

A-1 SHEET PILE DRIVING TEST

TP-14 TEST PIT

B-4 BORING

Reference: Drawing entitled "Puget Sound Power and Light Company, White River Hydroelectric Project, FERC Project No. 2494, Diversion Dam Rebuild, Dam Foundation and Excavation Plan, Drawing No. 7," by HDR Engineering, Inc., dated 05/21/92.

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SHEET PILE TEST LOCATIONS (A)

FIGURE 4

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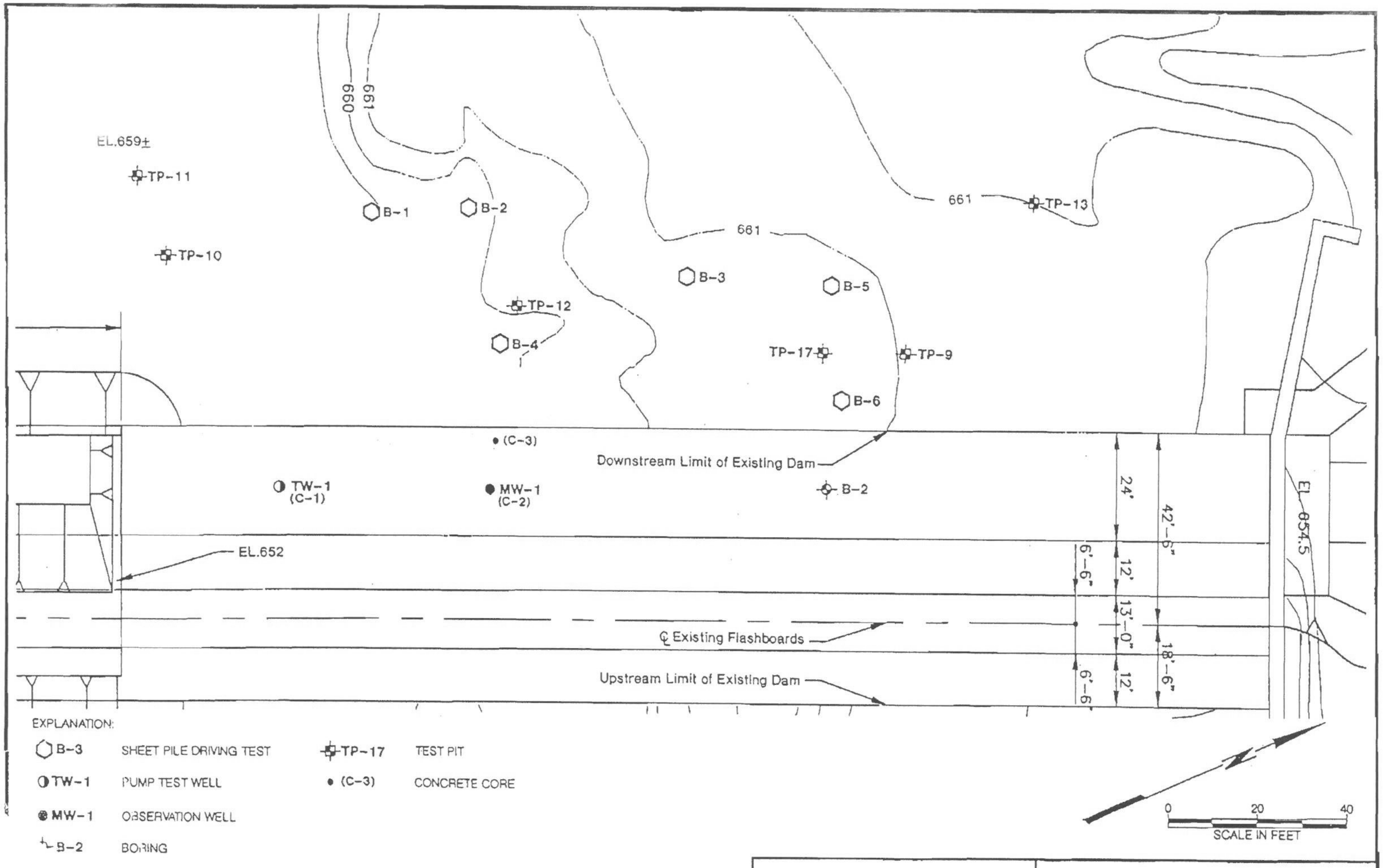
D-102



* Drawing entitled "Puget Sound Power and Light Company, White River Hydroelectric Project, FERC Project No. 2494, Diversion Dam Rebuild, Dam Foundation and ...vation Plan, Drawing No. 7," by HDR Engineering, Inc., dated 05/21/92.

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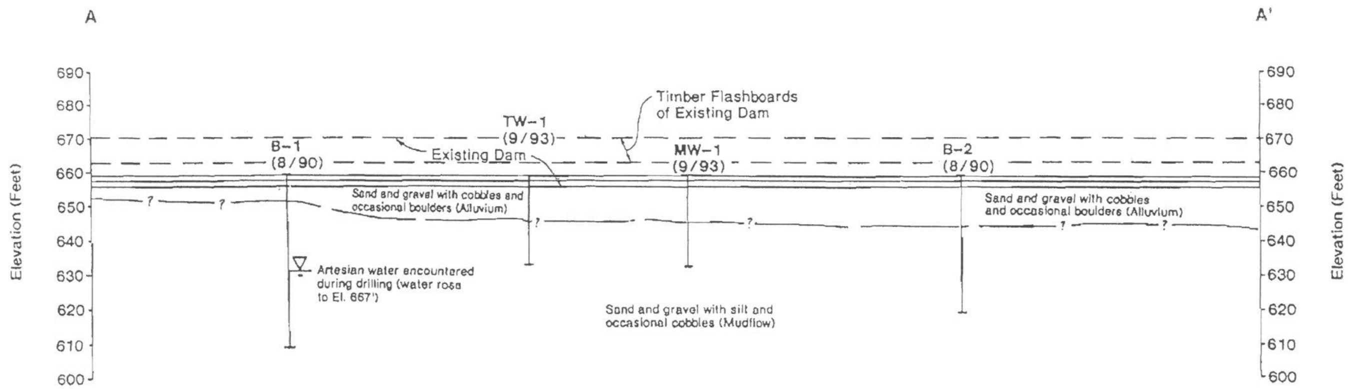
SHEET PILE TEST LOCATIONS (B)

FIGURE 5

0186-227-R06 DWM:BCH 10/27/93 Rev .../93

D-103



HORIZONTAL AND VERTICAL
SCALE IN FEET: 1" = 30'

EXPLANATION:

B-1 (8/90) — SUBSURFACE EXPLORATION (DATE DRILLED)

TW-1 – PUMP TEST WELL
MW-1 – OBSERVATION WELL
B-1 – BORING

Notes:
1. Refer to Figure 2 for location of profile.
2. Subsurface conditions shown on the profile are based on interpolation between widely spaced explorations and should be considered approximate.

Reference: Drawing entitled "Puget Sound Power and Light Company, White River Hydroelectric Project, FERC Project No. 2494, Diversion Dam Rebuild, Dam Foundation and Excavation Plan, Drawing No. 5." by HDR Engineering, Inc., dated 05/21/92.

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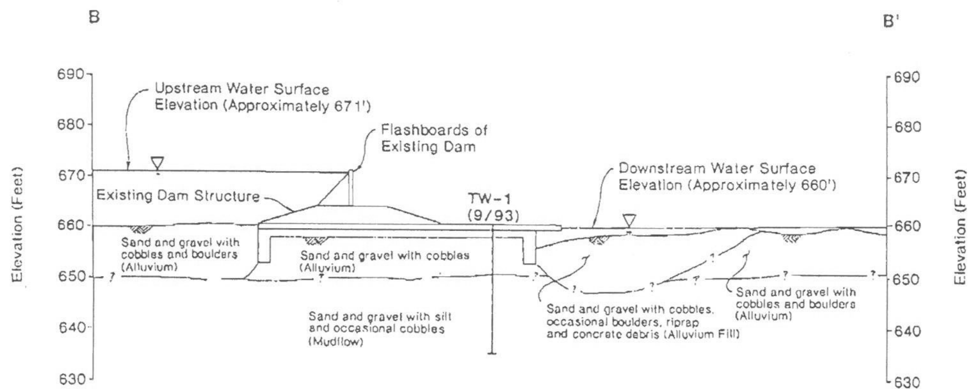
SUBSURFACE SOIL PROFILE A – A'

FIGURE 6

D-104

0386-227-R06 DAM.BDI 11/1/93 Rev 11/30/93 KSJ



HORIZONTAL AND VERTICAL
SCALE IN FEET: 1" = 20'

EXPLANATION:

TW-1 (9/93) — PUMP TEST WELL (DATE DRILLED)

Notes:
1. Refer to Figure 2 for location of cross section.
2. Subsurface conditions shown on the cross section are based on extrapolation of soil types encountered in TW-1 and should be considered approximate.

Reference: Drawing entitled "Puget Sound Power and Light Company, White River Hydroelectric Project, FERC Project No. 2494, Diversion Dam Rebuild, Dam Foundation and Excavation Plan, Drawing No. 7," by HDR Engineering, Inc., dated 05/21/92.

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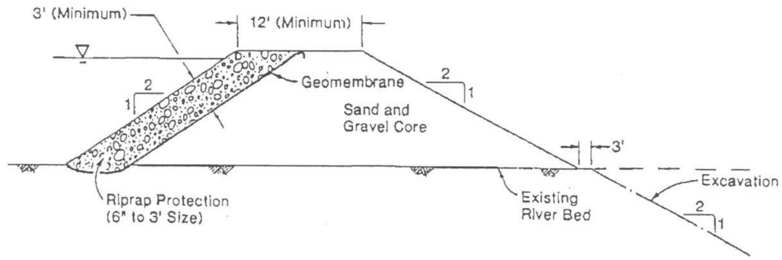
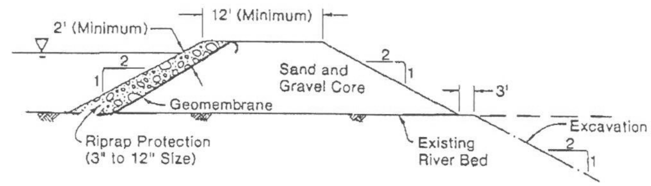
SUBSURFACE CROSS SECTION B - B'

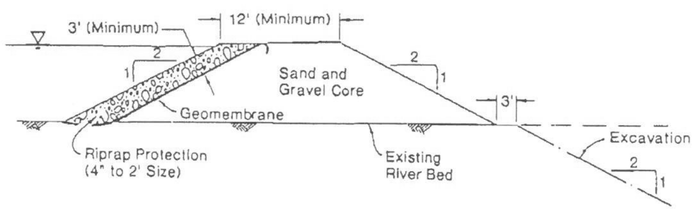
FIGURE 7



TYPE I COFFERDAM (HIGH ENERGY AREAS)
No Scale



TYPE III COFFERDAM (LOW ENERGY AREAS)
No Scale



TYPE II COFFERDAM (MODERATE ENERGY AREAS)
No Scale

Note: Refer to text for material specifications and discussion of cofferdam design.

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RECOMMENDATIONS FOR EARTH COFFERDAMS

FIGURE 8

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Subsurface explorations at the site were explored by drilling four borings from August 28 to September 4, 1990, excavating 8 test pits on September 14, 1990, excavating five test pits on August 2, 1991 and digging four hand holes on April 16, 1992. An additional two borings, MW-1 and TW-1, were drilled on September 9 and 10, 1993.

The explorations were either completed or were continuously monitored by a geotechnical engineer from our staff who selected sample intervals, examined and classified samples recovered, and kept a log of each boring based on examination of the samples. Exploration locations were measured by taping and pacing from existing survey markers located in the field. Ground surface elevations at the explorations have been interpreted from contours shown on a photogrammetric map constructed by Walker and Associates, Inc. entitled "White River," dated December 11, 1989.

The soils encountered in our explorations were classified visually in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is presented in Figure A-2. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. They also indicate the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples, it was interpreted.

BORINGS

The four borings designated as B-1 through B-4 were drilled to depths ranging from 40 to 50 feet using a truck-mounted air-rotary drill rig. Relatively undisturbed samples were obtained from the borings using a 3.25 inch O.D. split-barrel sampler driven into the soil using a 300-pound hammer falling a distance of approximately 30 inches. The number of blows required to drive the sampler the last 12 inches, or other indicated distances, is recorded on the boring logs. The locations of the borings are shown on the Site Plans, Figures I and 2.

Borings MW-1 and TW-1 were drilled by Holt Drilling, Inc. of Puyallup, Washington using a truck-mounted hollow-stem air rotary rig. The borings were completed as a monitoring well and a test well, respectively. Details of well installation are presented in the text of the report. Soil samples were typically obtained using a 3.25 inch OD (outer diameter) split-barrel sampler driven by a 300 pound hammer free falling. a distance of 30 inches. The number of blows required to drive the sampler the final 12 inches, or other indicated distances was recorded.

Soil sampling was also attempted using two other types of samplers. These include a 2.0 OD (outer diameter) split barrel sampler, driven using a 140 pound hammer, and a 5-inch OD split barrel sampler driven using a 500 pound hammer were also used. However, the recoveries obtained using these samplers were minimal and therefore their use discontinued.

The logs of the borings are presented in Figures A-3 through A-8.

TEST PITS

Test pits TP-1 through TP-8 were excavated in the east portion right bank dike with a rubber-tired backhoe to depths ranging from 7 to 10.5 feet. Test pits TP-9 through TP-17 were excavated in the river bed and left river bank using a track-mounted excavator. The depths of these nine test pits ranged from 4.5 to 16 feet below the river bed. The locations of the test pits are shown on the Site Plans in Figures 1 and 2 and in Figures 4 and 5.

The logs of the test pits are presented in Figures A-9 through A-16.

HAND HOLES

Hand holes H-1 through H-4 were excavated in sand bars in the river by digging with a hand shovel. The depth of the holes ranged from 1 to 3 feet. Location of the hand holes is shown in Figure 2. The logs of the hand holes are presented in Figure A-17.

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS More Than 50% Retained on No. 200 Sieve	GRAVEL More Than 50% of Coarse Fraction Retained on No. 4 Sieve	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND More Than 50% of Coarse Fraction Passes No. 4 Sieve	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS More Than 50% Passes No. 200 Sieve	SILT AND CLAY Liquid Limit Less Than 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY Liquid Limit 50 or More	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1. Field classification is based on visual examination of soil in general accordance with ASTM D2488-90.
2. Soil classification using laboratory tests is based on ASTM D2487-90.
3. Descriptions of soil density or consistency are based on interpretation of blow count data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water

Wet - Visible free water or saturated, usually soil is obtained from below water table

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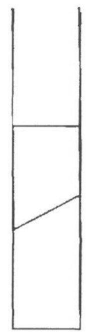
SOIL CLASSIFICATION SYSTEM

FIGURE A-1

LABORATORY TESTS:

AL	Atterberg limits
CP	Compaction
CS	Consolidation
DS	Direct shear
GS	Grain - size
%F	Percent fines
HA	Hydrometer analysis
SK	Permeability
SM	Moisture content
MD	Moisture and density
SP	Swelling pressure
TX	Triaxial compression
UC	Unconfined compression
CA	Chemical analysis

SOIL GRAPH:



SM Soil Group Symbol (See Note 2)

Distinct Contact Between Soil Strata

Gradual or Approximate Location of Change Between Soil Strata

Water Level

Bottom of Boring

BLOW-COUNT/SAMPLE DATA:

Blows required to drive a 2.4-inch I.D. split-barrel sampler 12 inches or other indicated distances using a 300-pound hammer falling 30 inches.

- 22 ■ Location of relatively undisturbed sample
- 12 ⊠ Location of disturbed sample
- 17 ▯ Location of sampling attempt with no recovery

Blows required to drive a 1.5-inch I.D. (SPT) split-barrel sampler 12 inches or other indicated distances using 140-pound hammer falling 30 inches.

- 10 ◨ Location of sample obtained in general accordance with Standard Penetration Test (ASTM D-1586) procedures
- 26 ◫ Location of SPT sampling attempt with no recovery

▤ Location of grab sample

"P" indicates sampler pushed with weight of hammer or against weight of drill rig.

NOTES:

1. The reader must refer to the discussion in the report text, the Key to Boring Log Symbols and the exploration logs for a proper understanding of subsurface conditions.

2. Soil classification system is summarized in Figure A-1.

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KEY TO BORING LOG SYMBOLS

FIGURE A-2

BORING B-1

TEST DATA

Surface Elevation (ft.) : 661.0

Depth in Feet	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow-Count	Samples	Group Symbol	DESCRIPTION
0							Rock and concrete filled timber crib dam
						GM	Dark gray sandy fine to coarse gravel with a trace of silt and occasional cobbles (dense, wet)
5				50/1"	☐		
	GS			50/5"	■	GP GM	Gray fine to coarse gravel with silt, sand and occasional cobbles (dense, wet)
10				50/6"	☒		
				50/5"	☐		Driving on a cobble
15				50/6"	☒		
	%F			50/6"	☒		
						SP SM	Dark gray gravelly medium to coarse sand with silt (dense, wet)
20				50/3"	☒		
25				50/6"	☒		
						GW	Dark gray fine to coarse gravel with sand, occasional cobbles and a trace of silt (dense, wet)
30	%F			50/4"	☒		
35	GS			50/4"	☒		
						GP GM	Gray sandy fine to coarse gravel with silt and occasional cobbles (dense, wet)
40							

Note: See Figure A-2 for explanation of symbols

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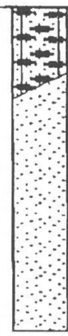
Log of Boring

FIGURE A-3 (1 OF 2)

BORING B-1
(Continued)

TEST DATA

Depth (feet)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow-Count	Samples	Group Symbol	DESCRIPTION
40	%F			50/4"	■		
43						SP	Dark gray medium to coarse sand with gravel (dense, wet)
45				50/4"	⊠		
50				50/5"	⊠		



Boring completed at 50.5 feet on 8/29/90
Ground water encountered at approximately 1 foot depth at time of drilling
Artesian water conditions encountered at 30 feet caused the water level in the boring to rise to about 5 feet above the top of boring during drilling

DEPTH IN FEET: 40, 45, 50, 55, 60, 65, 70, 75, 80

Note: See Figure A-2 for explanation of symbols

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Log of Boring

FIGURE A-3 (2 OF 2)

BORING B-2

TEST DATA

Surface Elevation (ft.): 661.0

Depth (ft)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Group Symbol	Description
0						Rock and concrete filled timber crib dam
~3					GW	Gray sandy fine to coarse gravel with occasional cobbles, a trace of silt and wood fragments (loose, wet)
~5				11		
~7.5	GS			42		Grades to medium dense
~9.5				50/5"		Driving on cobble
~10.5					GP GM	Dark grayish-brown sandy fine to coarse gravel with silt and occasional cobbles (dense, wet)
~12	%F			50/5"		
~15				50/2"		
~17				50/5"		Large piece of gravel in shoe
~18.5					GP GM	Gray silty, sandy fine to coarse gravel with cobbles (dense, wet)
~20	GS			50/5"		
~25				50/5"		
~30	%F			50/4"		Large piece of gravel in shoe
~35				50/4"		
~40	%F			50/4"		Boring completed at 40 feet on 8/30/90 Ground water encountered at approximately 1 foot depth at time of drilling

Note: See Figure A-2 for explanation of symbols

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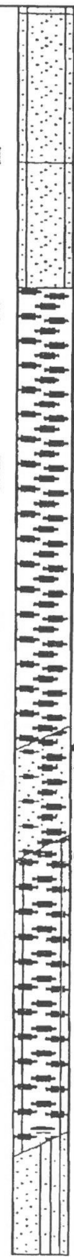
Log of Boring

FIGURE A-4

BORING B-3

TEST DATA

Surface Elevation (ft.): 673.0



Depth in Feet	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
0						SM	Dark brown silty fine to medium sand with gravel and occasional cobbles (loose, moist) (sod zone)
						SM	Dark brown silty fine to medium sand with gravel and occasional cobbles, (loose, moist)
5				8	■	SM	Dark brownish-gray silty fine sand (very loose, moist)
10				47	■	GP	Dark brownish-gray sandy fine to coarse gravel with occasional cobbles and a trace of silt (dense, wet)
15	%F			50/5" 100/5"	□ ⊠		
20	GS			90/6"	⊠		
						GW GM	Brown fine to coarse gravel with silt, sand and occasional cobbles, (dense, wet)
25	GS			81/6"	⊠		
						GM	Gray silty, sandy fine to coarse gravel with occasional cobbles (dense, wet)
30	%F			57/6" 50/5"	□ ■		
35				50/5"	⊠		
						SP SM	Gray medium sand with silt and occasional gravel (dense, wet)
40	%F			53/6"	■		

Boring completed at 40 feet on 8/31/90

Ground water encountered at approximately 12 feet depth at time of drilling and on 9/6/91

2-inch diameter monitor well installed to 25 feet on 8/31/90

Note: See Figure A-2 for explanation of symbols

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Log of Boring

FIGURE A-5

BORING B-4

TEST DATA

Lab Tests | Moisture Content (%) | Dry Density (pcf) | Blow-Count | Samples | Group Symbol

DESCRIPTION

Surface Elevation (ft.): 674.0

DEPTH IN FEET

Depth (ft)	Blow-Count	Group Symbol	Description
0		GP	Gray fine to coarse gravel with coarse sand and occasional cobbles (medium dense, wet)
5	50/5"		
		SP	Dark gray fine to coarse sand with gravel, occasional cobbles and a trace of silt (dense, wet)
10	50/5"		
		SP SM	Gray gravelly fine to coarse sand with silt and occasional cobbles (dense, wet)
	50/6"		
15			
20	60/6"		
25	62/5"		
30	60/5"		
35	50/4"	ML	Gray silt with sand and gravel (hard, wet)
		SP	Gray fine to coarse sand with gravel, occasional cobbles and a trace of silt (dense, wet)
40	75/2"		

Note: See Figure A-2 for explanation of symbols

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Log of Boring

FIGURE A-6 (1 OF 2)

TEST DATA

BORING B-4
(Continued)

Depth in Feet	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow-Count	Samples	Group Symbol	DESCRIPTION
40							
45				100/4"	⊠		
50				50/4"	⊠		Boring completed at 50 feet on 9/4/90 Ground water encountered at approximately 3 feet depth at time of drilling
55							
60							
65							
70							
75							
80							

Note: See Figure A-2 for explanation of symbols

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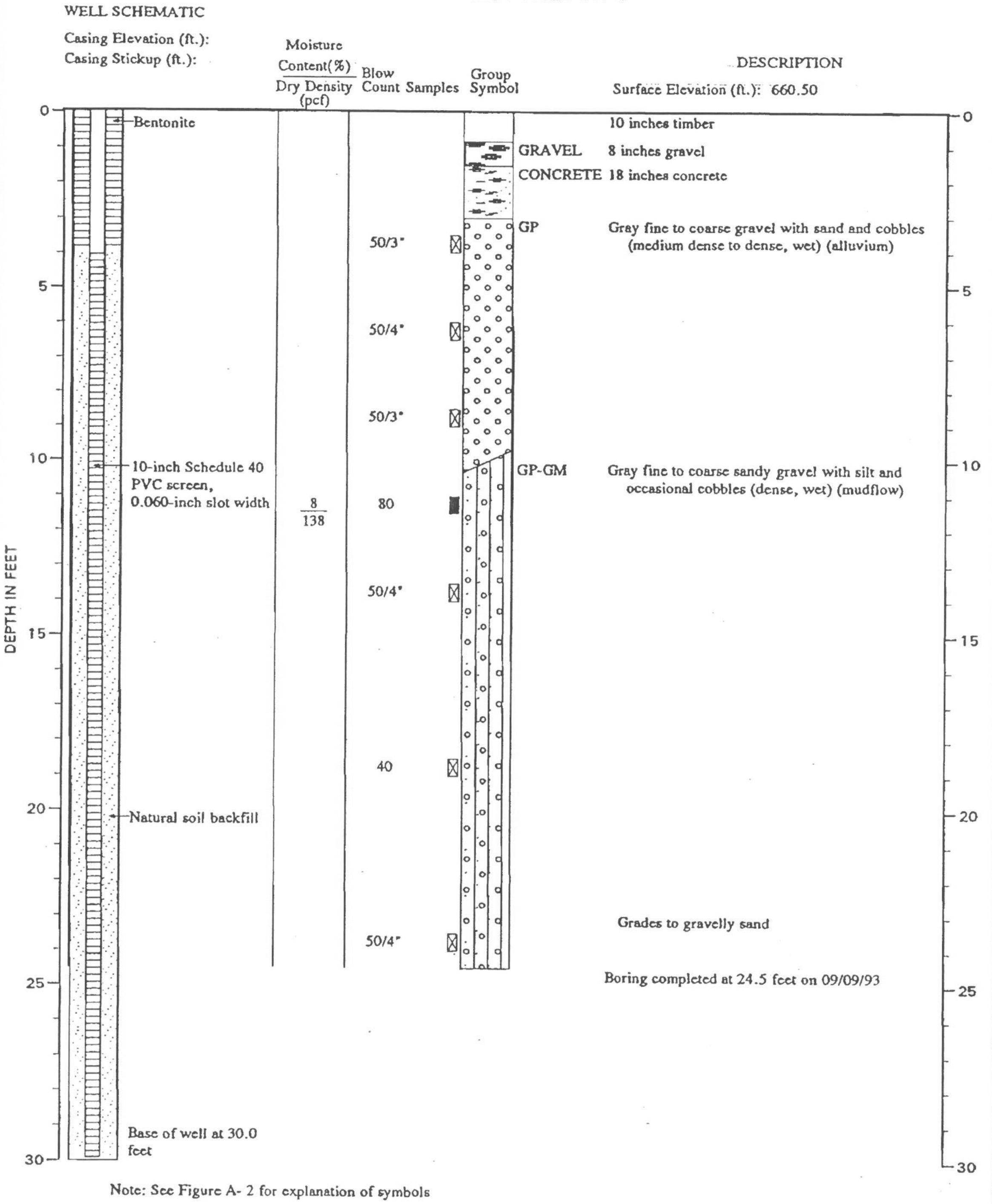
Log of Boring

FIGURE A-6 (2 OF 2)

TEST WELL TW-1



Note: See Figure A- 2 for explanation of symbols

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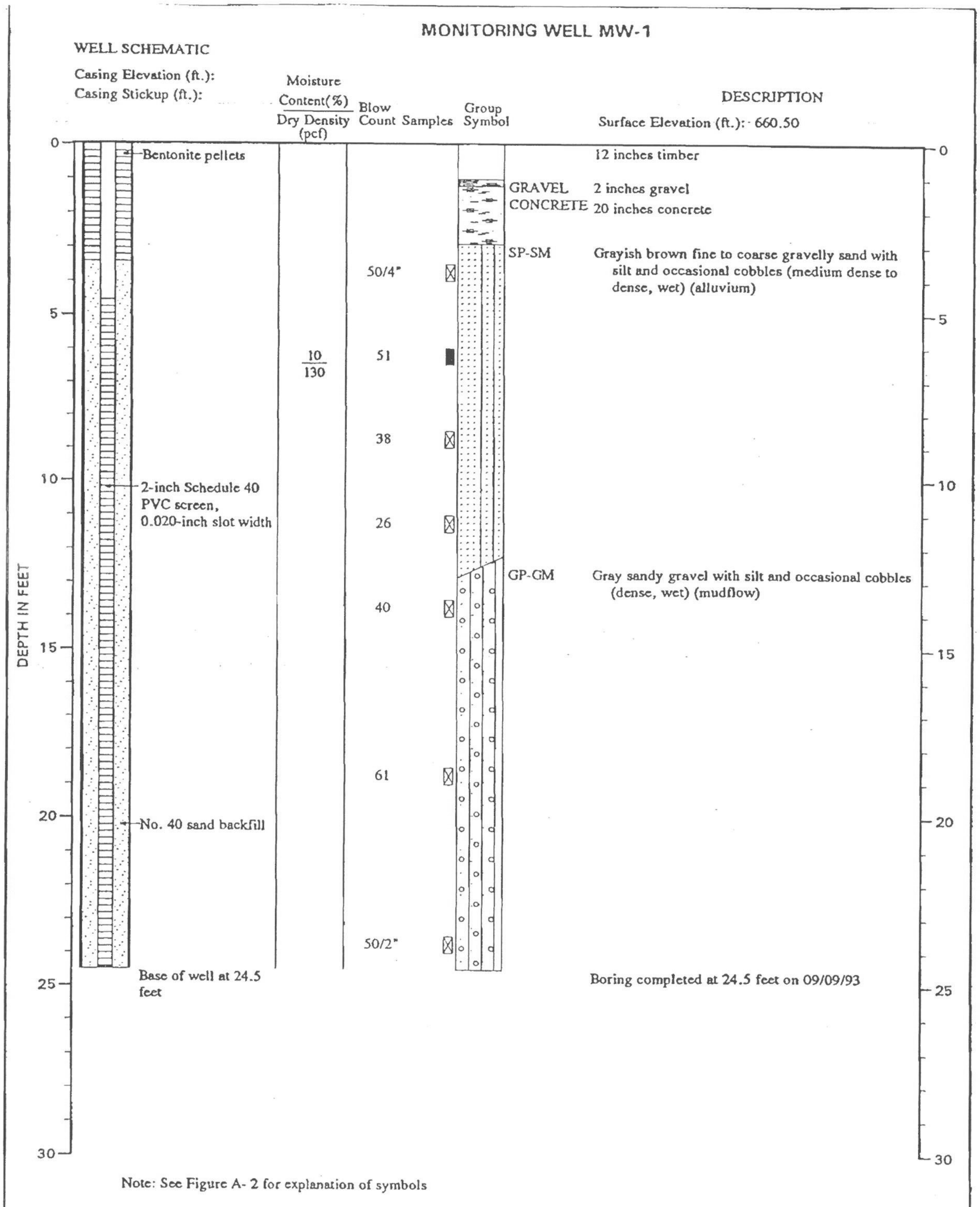
LOG OF TEST WELL

FIGURE A-7



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LOG OF MONITORING WELL

FIGURE A-8

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		<u>TEST PIT 1</u>
		Approximate elevation: 690 feet
0.0 - 0.5		4 inches minus rock spalls
0.5 - 2.0		12 inches minus rock spalls
2.0 - 4.0	SP	Dark grayish-brown gravelly fine to coarse sand with occasional cobbles, boulders and a trace of silt (medium dense, moist)
4.0 - 8.0	SP-SM	Gray gravelly fine to coarse sand with silt, occasional cobbles and small boulders (medium dense, moist)
8.0 - 10.0	SP	Dark grayish-brown gravelly fine to coarse sand with occasional cobbles, small boulders and a trace of silt (medium dense, moist)
		Test pit completed at 10.0 feet on 9/14/90
		No ground water seepage encountered
		Disturbed sample obtained at 2.5 feet
		<u>TEST PIT 2</u>
		Approximate elevation: 689 feet
0.0 - 0.5	GP	Gray fine to coarse gravel with sand (medium dense, moist)
0.5 - 1.5		12 inch minus rock spalls
1.5 - 10.5	SP-SM	Brown gravelly fine to medium sand with silt, occasional cobbles and small boulders (loose, moist)
		Test pit completed at 10.5 feet on 9/14/90
		No ground water seepage encountered
		Disturbed sample obtained at 10.0 feet
		Large boulder encountered at 10.5 feet

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-9

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		TEST PIT 3
		Approximate elevation: 685 feet
0.0 - 0.5	SP-SM	Brown fine to medium sand with silt, gravel and roots (loose, moist) (sod zone)
0.5 - 6.0	GP	Dark grayish-brown sandy fine to coarse gravel with occasional cobbles, small boulders and roots to 2 feet in depth (loose, moist) (fill)
6.0 - 8.0	GP	Dark grayish-brown sandy fine to coarse gravel with occasional cobbles and small boulders
		Test pit completed at 8.0 feet on 9/14/90
		No ground water seepage encountered
		Large boulder encountered at 8.0 feet
		TEST PIT 4
		Approximate elevation: 688 feet
0.0 - 0.5	SP-SM	Brown fine to medium sand with silt and gravel (loose, moist) (sod zone)
0.5 - 8.0	SP	Dark gray gravelly medium to coarse sand (loose, moist) (fill)
		Test pit completed at 8.0 feet on 9/14/90
		No ground water seepage encountered
		Moderate caving below 5.0 feet

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A–10

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		TEST PIT 5
		Approximate elevation: 683 feet
0.0 - 0.5	SP-SM	Brown fine to medium sand with silt and gravel (loose, moist) (sod zone)
0.5 - 7.5	GP	Dark gray sandy fine to coarse gravel with occasional cobbles, small boulders and a trace of silt (loose, moist) (fill)
		Test pit completed at 7.5 feet on 9/14/90
		No ground water seepage encountered
		Wood debris at 4.0 feet
		Moderate caving below 5.0 feet
		TEST PIT 6
		Approximate elevation: 687 feet
0.0 - 0.5	SP-SM	Brown fine to medium sand with silt, gravel and roots (loose, moist) (sod zone)
0.5 - 1.0		12 inches minus rock spalls
1.0 - 6.0	GP	Dark gray sandy fine to coarse gravel with occasional cobbles, small boulders and a trace of silt (loose, moist) (fill)
6.0 - 10.0	SP	Dark gray gravelly fine to coarse sand with occasional cobbles, small boulders and a trace of silt (loose, moist) (fill)
		Test pit completed at 10.0 feet on 9/14/90
		No ground water seepage encountered
		Large boulder encountered at 10.0 feet
		TEST PIT 7
		Approximate elevation: 685 feet
0.0 - 5.0	SP	Gray fine to medium sand with gravel, occasional cobbles, small boulders and roots to 2.0 feet (loose, moist) (fill)
5.0 - 8.0	SP	Gray fine to medium sand with occasional gravel (loose, moist)
		Test pit completed at 8.0 feet on 9/14/90
		No ground water seepage encountered

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-11

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		<u>TEST PIT 8</u>
		Approximate elevation: 682 feet
0.0 - 3.5	SP	Gray gravelly fine to medium sand with occasional cobbles and roots to 2 feet (loose, moist) (fill)
3.5 - 6.0	SP	Gray gravelly fine to coarse sand with occasional cobbles, small boulders and a trace of silt (loose, moist)
		Test pit completed at 6.0 feet on 9/14/90
		No ground water seepage encountered

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-12

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		TEST PIT TP-9 (In River)
0.0 - 11.5	GW	Dark brown fine to coarse gravel with fine to coarse sand, numerous cobbles to 12 inches (15 to 20 percent by weight), occasional boulders to 24 inches (5 to 10 percent by weight), tree roots and limbs to 4 inches in diameter, occasional concrete pieces (loose to medium dense, moist to wet)
11.5 - 13.0	GW-GM	Gray fine to coarse gravel with silt, fine to coarse sand and occasional cobbles to 12 inches (medium dense, to dense, wet) (Osceola mudflow)
		Test pit completed at 13.0 feet on 08/02/91
		Gravel, cobbles and boulders are subrounded
		Ground surface covered with layer of boulders to 18 inches
		Test pit walls sloughing, difficult to obtain sample of mudflow
		TEST PIT TP-10 (In River)
		Water depth: 1.5 feet at test pit
0.0 - 12.0	GW	Dark brown fine to coarse gravel with fine to coarse sand, numerous cobbles to 6 inches (10 to 15 percent by weight), occasional cobbles to 12 inches (less than 5 percent by weight), occasional boulders to 24 inches (less than 1 percent by weight), numerous wood pieces, tree roots and limbs to 6 inches in diameter (loose to medium dense, wet)
		Test pit completed at 12.0 feet on 08/02/91
		Refusal on large obstruction at 12 feet (slab or boulder), moved test pit 15 feet downstream (Test Pit 2A)
		Gravel, cobbles and boulders are subrounded

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-13

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		TEST PIT TP-11 (In River)
		Water depth: 1.5 feet at test pit
0.0 - 16.0	GW	Dark brown fine to coarse gravel with fine to coarse sand, numerous cobbles to 6 inches (10 to 15 percent by weight), occasional cobbles to 12 inches (less than 1 percent by weight), few boulders to 14 inches (less than 1 percent by weight), numerous tree roots and limbs to 6 inches in diameter, occasional metal pieces (loose to medium dense, wet)
		Test pit completed at 16.0 feet on 08/02/91
		Caving severely, digging stopped at 16-foot depth
		Gravel, cobbles and boulders are subrounded
		TEST PIT TP-12 (In River)
		Water depth: 1 foot at test pit
0.0 - 15.0	GW	Dark brown fine to coarse gravel with fine to coarse sand, numerous cobbles to 6 inches (10 to 15 percent by weight), no boulders, occasional tree roots and limbs to 3 inches in diameter (loose to medium dense, wet)
		Test pit completed at 15.0 feet on 08/02/91
		Severe caving stopped digging at 15.0 feet
		Gravel and cobbles are subrounded
		TEST PIT TP-13 (In River)
		Ground surface: 1 foot higher than dam apron
0.0 - 7.0	GW	3-inch to 12-inch cobbles in dark brown fine to coarse sand and fine to coarse gravel matrix, boulders to 24 inches (20 to 25 percent by weight) (loose to medium dense, moist to wet)
		Test pit completed at 7.0 feet on 08/02/91
		Caving severely at 7.0 feet, can't keep hole open below 7.0 feet depth
		Gravel, cobbles and boulders are subrounded
		Surface covered by boulders to 24 inches
		Water observed at 6.5 feet

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-14

GEI 110-103

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		TEST PIT 14 (in River)
0.0 - 4.5	GP	Brown-black sandy gravel with cobbles and boulders to 18 inches diameter
		Test pit completed at 4.5 feet on 11/02/93
		Depth of river water at test pit location is 4.5 feet
		No caving encountered
		No samples obtained
		TEST PIT 15
0.0 - 3.0	SP-GP	Brown fine to medium sand with gravel, cobbles and occasional boulders to 12 inch diameter (dense to very dense, moist) (fill)
3.0 - 5.5	SM	Gray silty sand with gravel and cobbles (very dense, moist to wet)
		Test pit completed at 5.5 feet on 09/12/93
		Slight ground water seepage encountered at 3.0 feet
		No caving encountered
		Disturbed soil samples obtained at 2.0 and 5.0 feet
		TEST PIT 16
0.0 - 4.0	SP	Brown fine to medium sand with gravel, cobbles and a trace of silt and scattered roots (medium dense, moist to dry)
4.0 - 10.0	SP	Dark brown fine to medium sand with gravel and cobbles (medium dense, moist)
		Test pit completed at 10.0 feet on 09/10/93
		No ground water seepage encountered
		No caving encountered
		Disturbed soil samples obtained at 4.0 and 8.0 feet

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-15

LOG OF TEST PIT

DEPTH BELOW GROUND SURFACE (FEET)	SOIL GROUP CLASSIFICATION SYMBOL	DESCRIPTION
		TEST PIT 17
0.0 - 5.0	GP	Brown-black sandy gravel with cobbles, boulders to 18 inches diameter and concrete debris to 5 feet diameter (very dense, wet)
		Excavator bucket refusal on concrete debris at 5.0 feet on 11/02/93
		Depth of river water at test pit location is 4.5 feet
		No ground water seepage encountered
		No caving encountered

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.

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LOG OF TEST PIT

FIGURE A-16

LOG OF HAND HOLES

(See Figure 2 for Location)

Hole No.	Depth Interval (inches)	U.S.E.	Soil Description
H-1	0 - 6	SM	Silty fine sand with a trace of organic material
	6 - 18	GP-GM	Coarse gravel with soft and fine sand
			Hand hole completed on 4/16/92
			Soil samples obtained at 6 and 18 inches
			Hole did not fill with water
H-2	0 - 6	SP-SM	Fine sand with silt
	6 - 18	SM	Silty fine sand
	18 - 36	SM	Silty fine sand with fine to coarse gravel
			Hand hole completed on 4/16/92
			Soil samples obtained at 6, 18 and 36 inches
			Hole filled slowly with water
H-3	0 - 12	GP	Coarse gravel with medium sand
			Hand hole completed on 4/16/92
			Soil sample obtained at 12 inches
			Hole filled rapidly with water
H-4	0 - 12	GP	Medium sandy fine to coarse gravel with occasional fine to coarse sand
			Hand hole completed on 4/16/92
			Soil samples obtained at 12 inches
			Hole filled rapidly with water

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LOG OF HAND HOLES

FIGURE A–17

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

GEOTECHNICAL INDEX TESTS

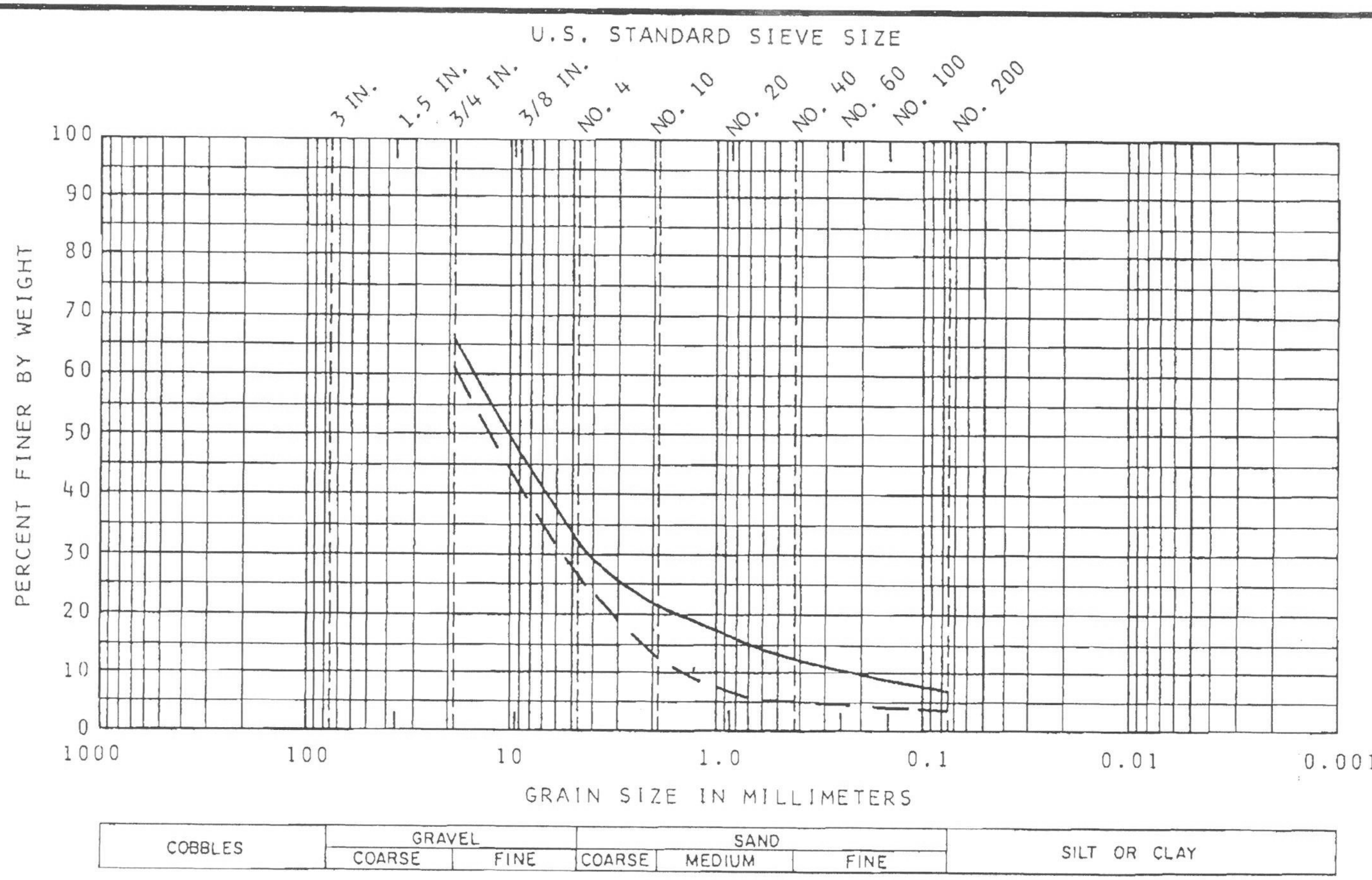
All soil samples were brought to our laboratory for further examination. Selected samples were tested to determine grain size characteristics. Mechanical grain-size analyses were performed on eighteen representative soil samples. Gradation curves for these samples are presented in Figures B-1 through B-9. Twelve additional soil samples were tested for percent fines (material passing the number 200 sieve). The percent fines test results are presented in Figure B-10. The results from moisture content and density tests performed on selected soil samples are shown on the boring logs.

SLUG TESTS

A series of slug tests was performed on the 2-inch-diameter well installed within boring B-3. Slug test numbers 1 and 2 were performed by adding a one and four gallon slug of water to the well, respectively. A solid 1.25-inch-diameter rod was used as the slug for test numbers 3 and 4. The rod was inserted and removed from the well for the respective tests. Water level measurements were measured and recorded with a data acquisition system consisting of a pressure transducer, a data logger, and a portable personal computer. The field data was reduced using Hvorslev's method and the Bouwer-Rice method. The results are presented in Figure B-11.

COMPRESSIVE STRENGTH TESTS OF CONCRETE CORES

GeoEngineers photographed and visually examined the concrete cores obtained from the existing dam. Three cores, C-1, C-2 and C-3 were submitted to an outside testing company for compressive strength tests. Cores C-1 and C-2 were tested and broke at a compressive strength of 3530 and 4450 psi (pounds per square inch), respectively. Reinforcing steel was encountered in core C-3 and therefore could not be tested.



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
———	B-1 SAMPLE 1	7'	FINE TO COARSE GRAVEL WITH SILT AND SAND (GP-GM)
— — —	B-1 SAMPLE 9	3.5'	FINE TO COARSE GRAVEL WITH SAND (GW)

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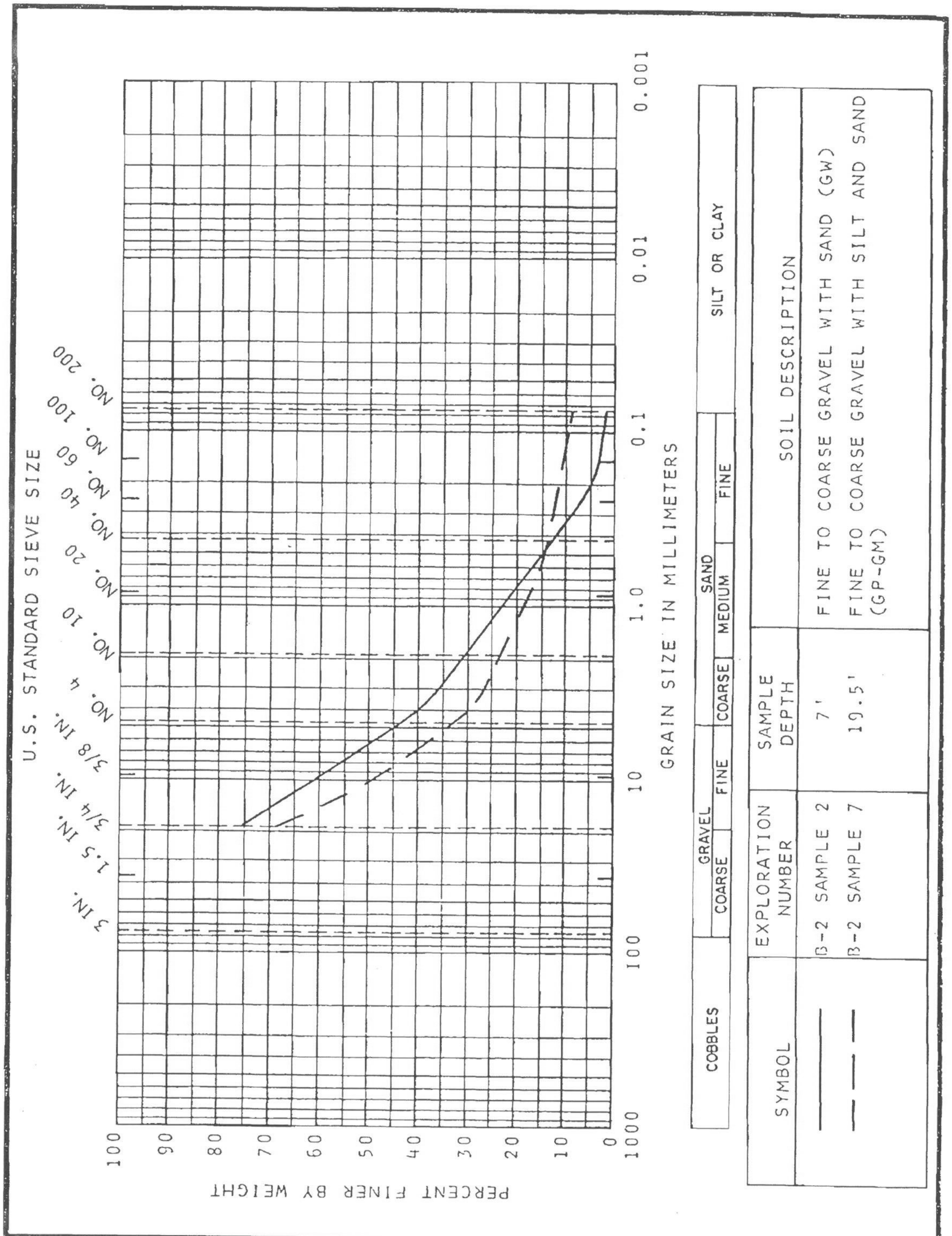
GRADATION CURVES

FIGURE B-1

G 50-88 0186-115-B06 DJC:BDH 10.17.90



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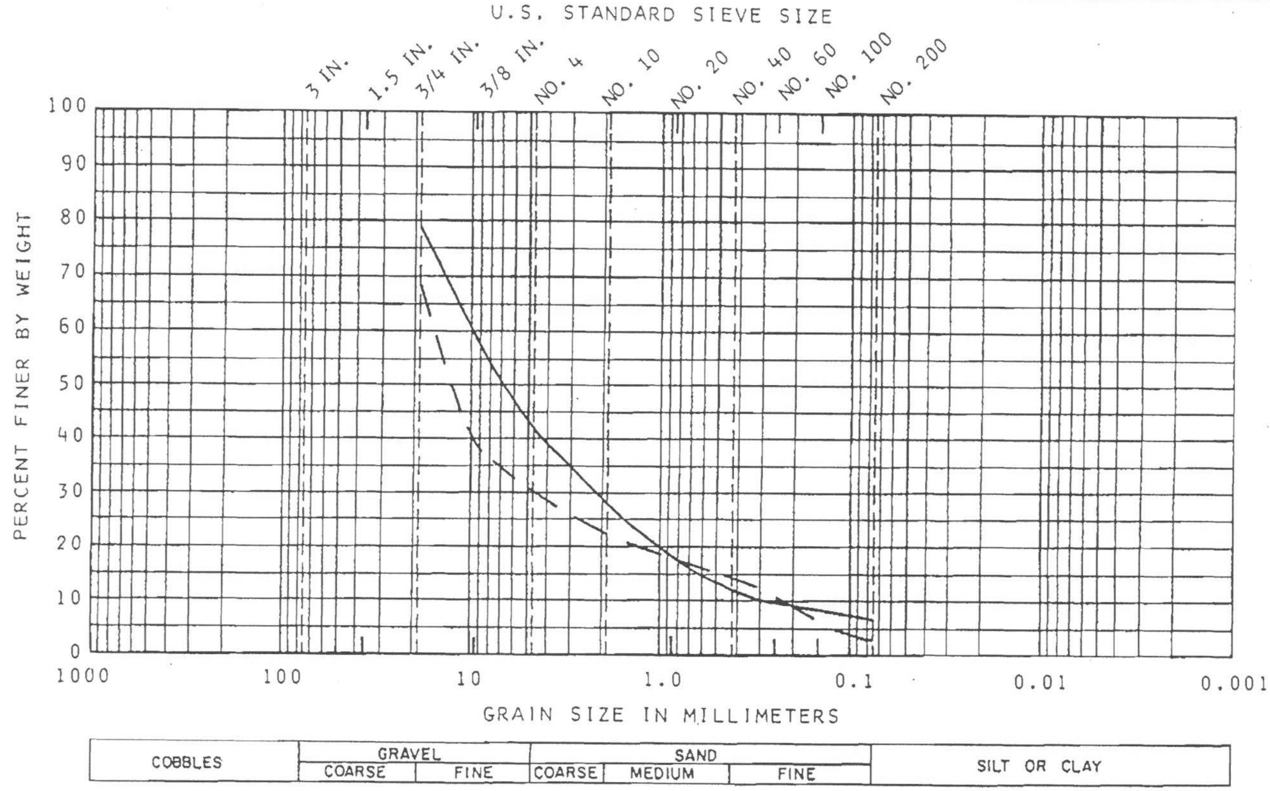
GRADATION CURVES

FIGURE B-2

G[r]G 30-88 0186-115-B06 DJC:BDH 10.17.90



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
————	B-3 SAMPLE 5	24.5'	FINE TO COARSE GRAVEL WITH SILT AND SAND (GW-GM)
— — —	B-3 SAMPLE 4 CUTTINGS	19.5'	FINE TO COARSE GRAVEL WITH

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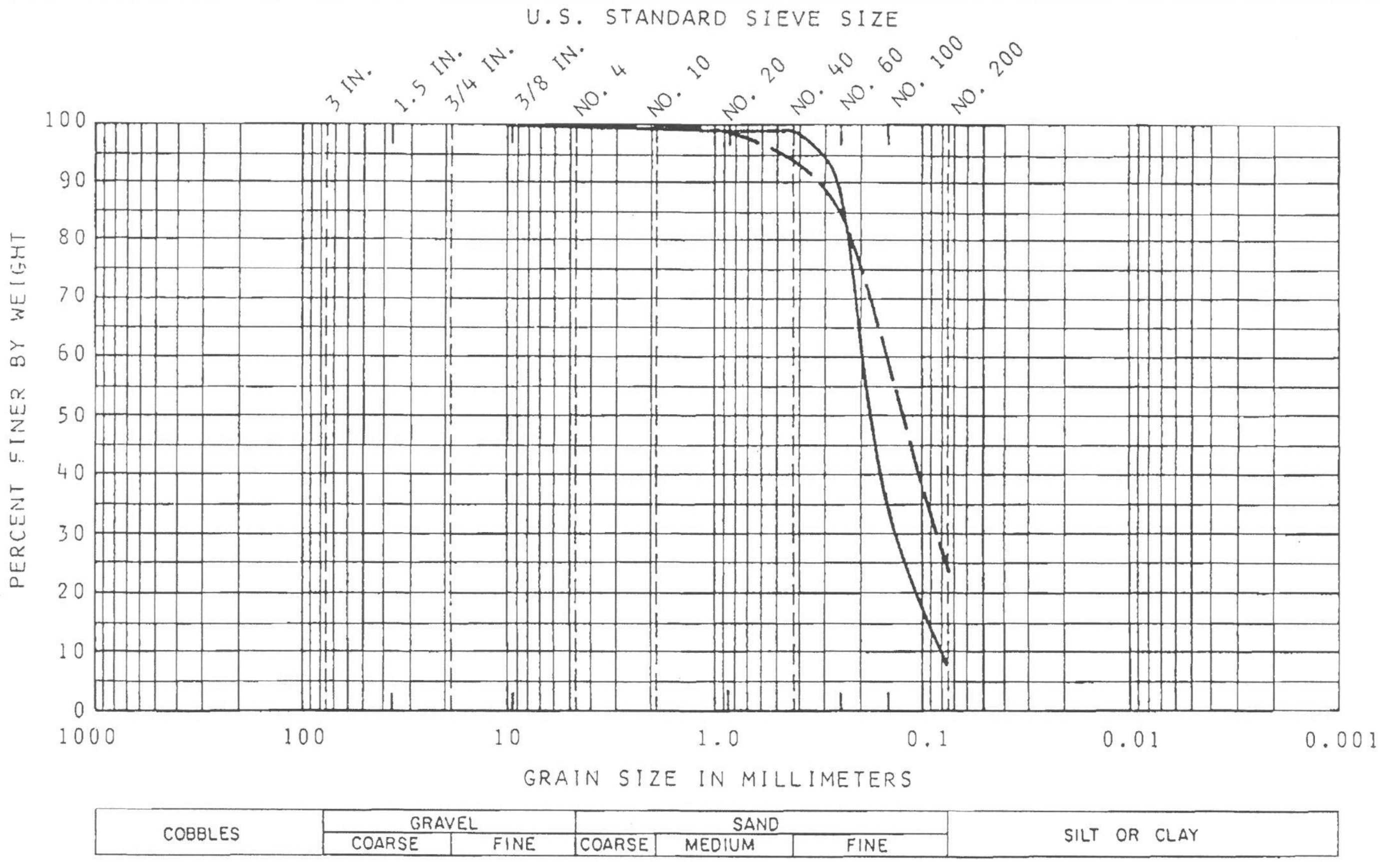
GRADATION CURVES

FIGURE

30-88

0186-115-R06



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
———	H-2	6"	FINE SAND WITH SILT (SP-SM)
— — —	H-2	18"	SILTY FINE SAND (SM)

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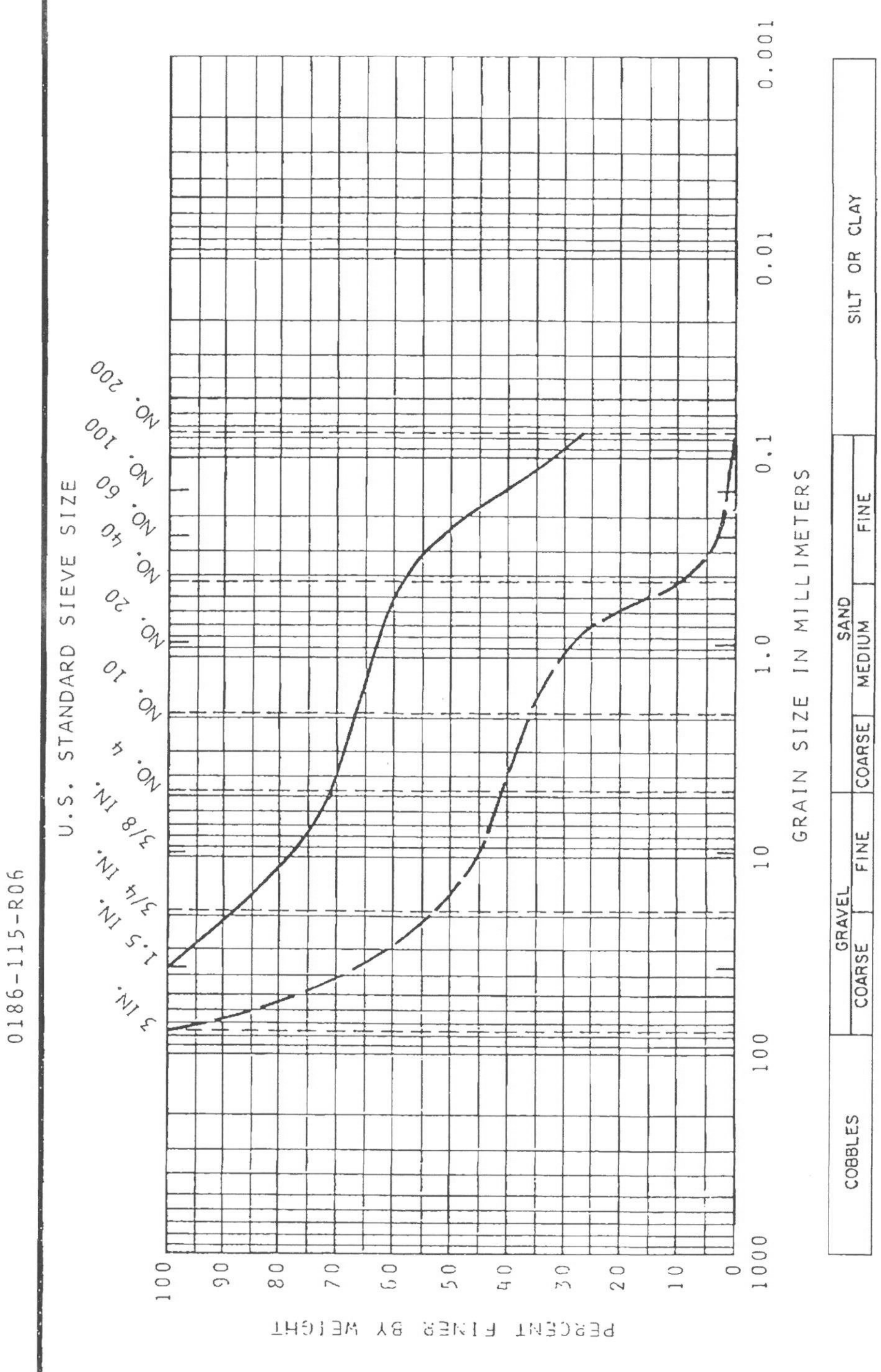
GRADATION CURVES

FIGURE B-5

0186-115-R06



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
	H-2	36"	SITLY FINE SAND WITH FINE TO COARSE GRAVEL (SM)
	H-3	12"	COARSE GRAVEL WITH MEDIUM SAND (GP)

30-88

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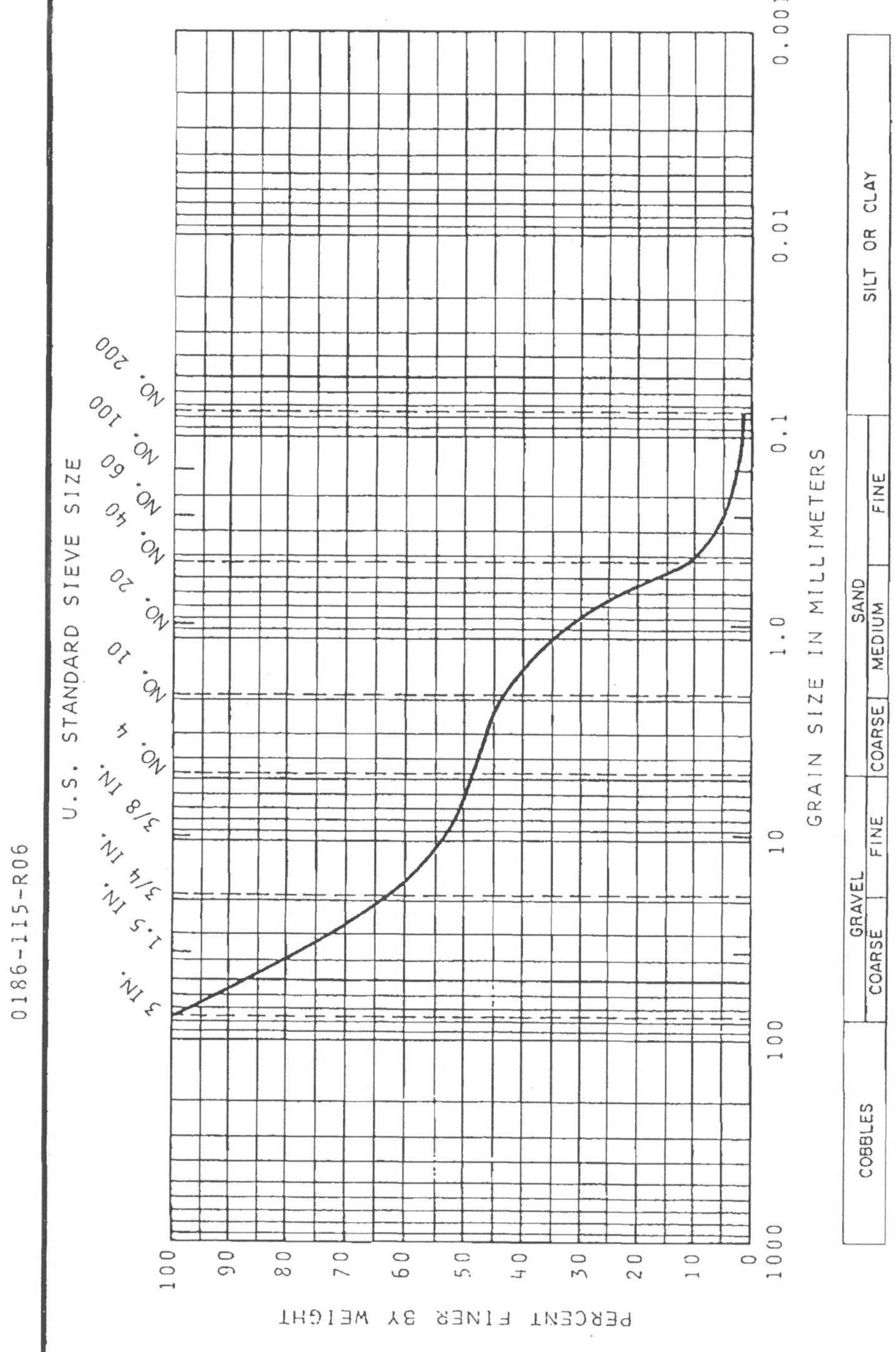
GRADATION CURVES

FIGURE B-6

0186-115-R06



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
——	H-4	12"	MEDIUM SANDY FINE TO COARSE GRAVEL WITH OCCASIONAL FINE TO COARSE SAND (GP)

30-88

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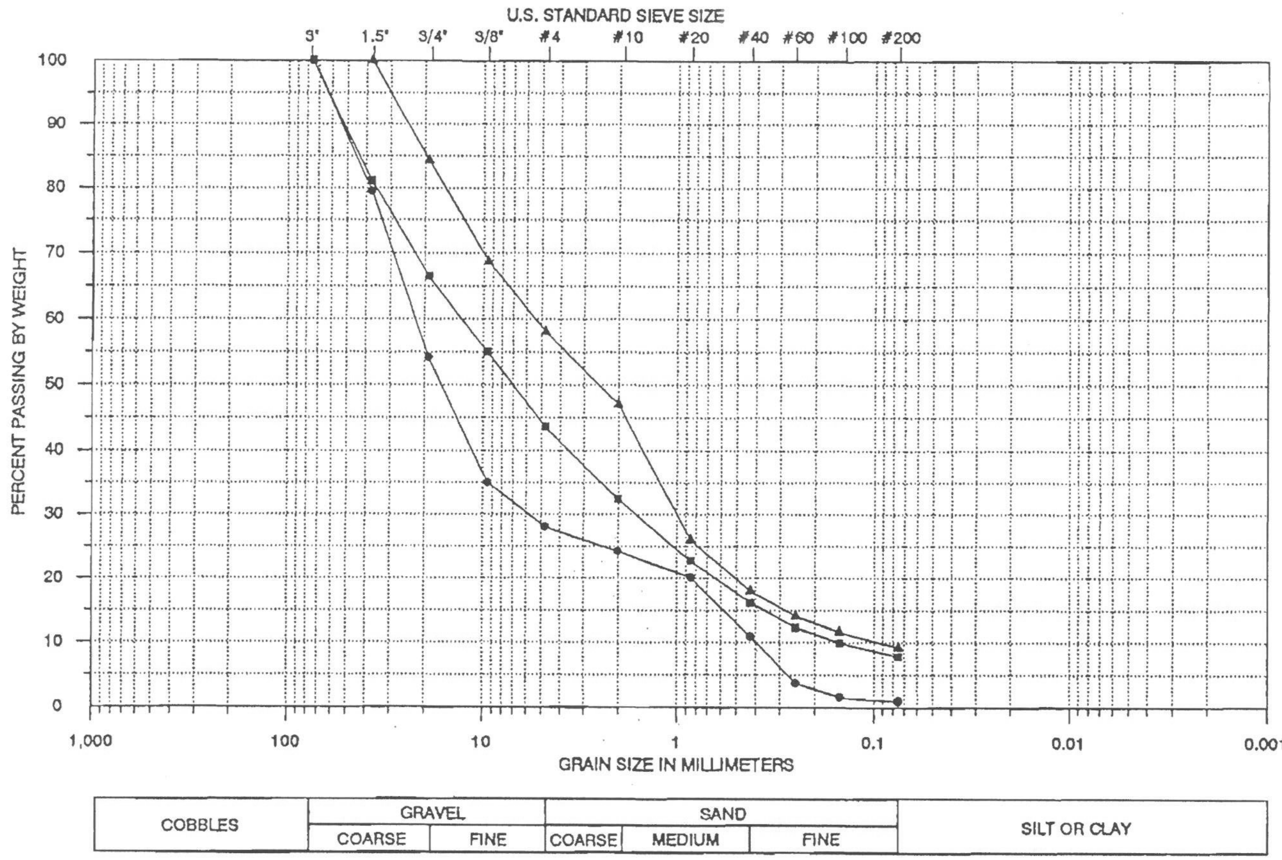
GRADATION CURVES

FIGURE B-7

0186-227· SBM:DWM:GMD:vvv 11/17/93 (186227B8)



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	SOIL DESCRIPTION
●	TW-1 S3	8.5	Brown fine to coarse gravel with fine to medium sand (GP)
■	TW-1 S6	18.5	Gray fine to coarse sandy fine to coarse gravel with silt (GP-GM)
▲	TW-1 S7	23.5	Gray fine to coarse gravelly fine to coarse sand with silt (SP-SM)

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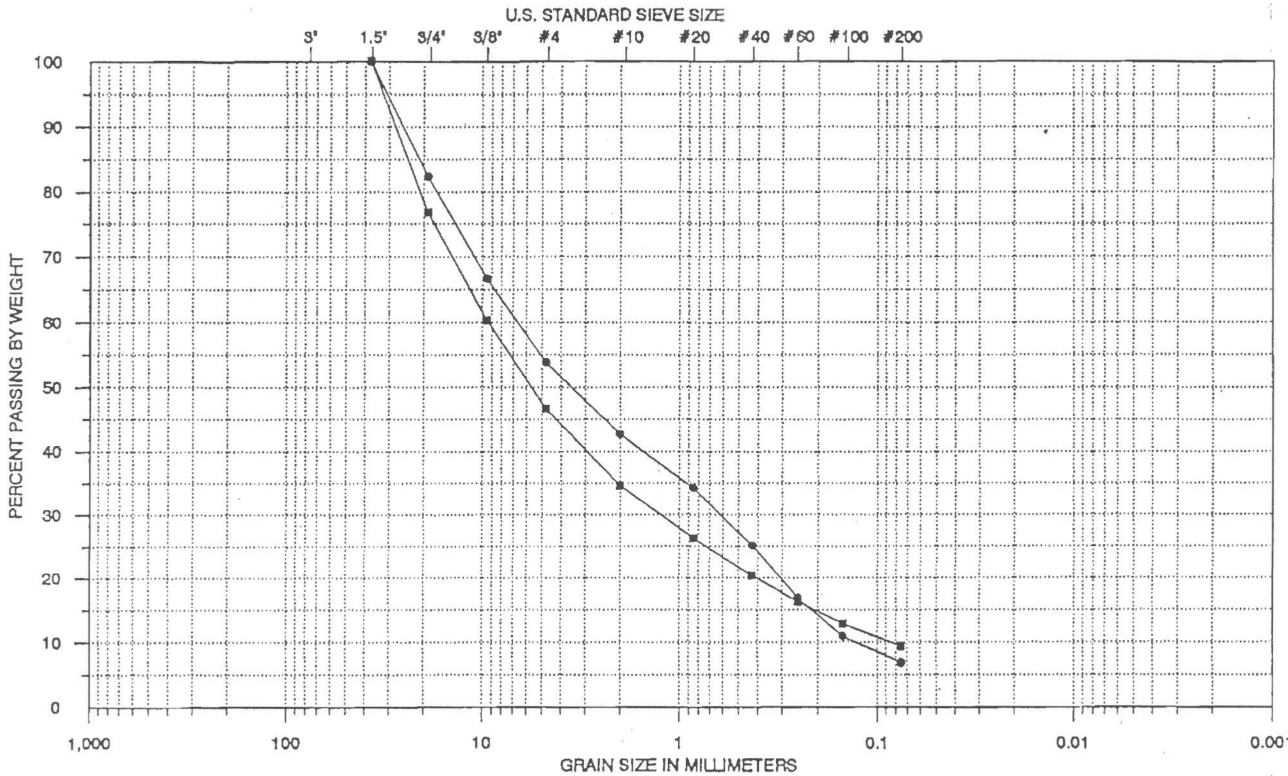
GRADATION CURVES

FIGURE B-8

0186-227 . .d SBM:DWM:GMD:vvv 11/17/93 (186227B9)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	SOIL DESCRIPTION
●	MW-1 S1	3.5	Gray fine to coarse gravelly fine to coarse sand with silt (SP-SM)
■	MW-1 S5	13.5	Brown fine to coarse sandy fine to coarse gravel with silt (GP-GM)

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GRADATION CURVES

FIGURE B-9

PERCENT FINES DATA

Boring Number	Depth of Sample (feet)	Sample Description	Percent Fines (%)
B-1	17.0	Fine to coarse gravel with a trace of silt (GP)	4.9
B-1	30.0	Fine to coarse gravel with sand (GW)	0.8
B-1	40.0	Fine to coarse gravel with silt and sand (GP-GM)	7.4
B-2	12.0	Fine to coarse gravel with silt and sand (GP-GM)	5.2
B-2	29.5	Fine to coarse gravel with sand and a trace of silt (GP)	4.4
B-2	39.5	Fine to coarse gravel with silt and sand (GP-GM)	7.5
B-3	15.0	Fine to coarse gravel with sand and a trace of silt (GP)	3.0
B-3	30.0	Silty fine to coarse gravel with sand (GM)	16.6
B-3	39.5	Fine to coarse gravel with silt and sand (GP-GM)	5.9
TW-1	14.0	Fine to coarse sandy gravel with silt (GP-GM)	10.4
MW-1	8.5	Fine to coarse gravelly sand with silt (SP-SM)	5.9
MW-1	23.0	Fine to coarse sandy gravel with silt (GP-GM)	5.4

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PERCENT FINES DATA

FIGURE B-10

SLUG TEST RESULTS

Slug Test Number	Permeability (cm/sec)	
	Hvorslev's Method	Bower-Rice Method
1	1.2 x 10-2	1.0 x 10-2
2	3.4 x 10-2	2.4 x 10-2
3	8.2 x 10-3	6.4 x 10-3
4	2.6 x 10-2	1.7 x 10-2

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SLUG TEST RESULTS

FIGURE B-11

APPENDIX C

SHEET PILE DRIVING RECORDS

TABLE C-1
SHEET PILE DRIVING RECORD - LOCATION A

Test Location[1]	Location	Hammer Frequency[2] (rmp)	Depth Driven Below Water[3]	Time		• Comments
				Min	Sec	
A-1	Offshore	--	0 - 4-1/2'			Water
(First		1,000	4-1/2' - 6'	0	27	Difficult driving
Attempt)		1,500	6 - 6-1/2'	3	10	• Very difficult driving
						• Pile could not be driven plumb
						• Unable to drive below 6-1/2'.
						• Lower 2' of pile is severely worn
A-1	Offshore	1,500	0 - 4-1/2'	--	--	Water
(Second		1,500	4-1/2' - 6-1/2'	8	27	• Very difficult driving below 6-1/2'
Attempt)		1,500	6-1/2' - 6-3/4'	0	47	• Unable to maintain plumb
		1,500	6-3/4' - 7'	1	41	• Pile appears to be "bouncing" on a large cobble/boulder
		1,500	7' - 8'	4	36	• Lower 4-1/2' of pile is severely worn
		1,500	8' - 9'	3	43	• Upper 1' of pile is bent due to hammer weight
A-2	Offshore	1,500	0 - 4-1/2'			Water
		1,500	4-1/2' - 6'	4	1	• Very difficult driving
		1,500				• Unable to maintain plumb
A-3	Onshore	1,500	0 - 3'	4	3	• Very difficult driving
						• Pile appeared to bounce on a large cobble and boulder
						• 6 - 8" diameter cobbles were removed from the upper 1 foot of the ground surface
						Unable to drive piles below 3'

Notes:
[1]See Figure 4 for general sheet pile driving test location.
[2]Hammer type: Tunkers 60.05 vibratory hammer rated at 66 tons drive force.
[3]Pile type: Pair of 3/8" wall thickness. Z sheets. Pile tips were used during driving at all locations except A-1 (first attempt).

TABLE C-2
SHEET PILE DRIVING RECORD - LOCATION B

Test Location[1]	Hammer Type[2]	Hammer Frequency (rmp)	Depth Driven Below Water[3]	Time		Comments
				Min	Sec	
B-1 (First Attempt)	Tunkers 60.05	1,500	0 - 1-1/2' 1-1/2' - 9-1/4'	-- 4	-- 49	Water • Pile appears to bounce on a boulder • The lower 7-1/2' of pile is completely worn • Unable to drive pile plumb
B-2	Tunkers 60.05	1,500	0 - 1-1/2' 1-1/2' - 5'	-- 2	-- 17	Water • Pile sways severely at 5' during driving • Pile appears to bounce on a large cobble/boulder at 5' depth • Unable to drive below 5'
B-3	Tunkers 60.05	1,500	0 - 1' 1' - 6-1/2'	-- 3	-- 58	Water • Unable to maintain plumb below 5' • The pile appears to bounce on a large boulder at 6-1/2' • Unable to drive below 6'
B-4	Tunkers 60.05	1,500	0 - 1/2' 1/2' - 3'			Water • The pile appears to bounce severely on a large boulder at 3' • Unable to maintain plumb
B-5[4]	Tunkers 60.05	1,500	0 - 4-1/2' 4-1/2' - 6-3/4'	-- 2	-- 44	Water • Unable to maintain plumb • Unable to drive below 6-3/4'
B-6[4]	Tunkers 60.05	1,500	0 - 3-1/2' 3-1/2' - 5-1/2'	 1	 44	Water • The pile appears to bounce severely on a large boulder at 5-1/2' • Unable to maintain plumb
B-1[4] (Second Attempt)	MKT V-30	1,600	0 - 4-1/2' 4-1/2' - 9-1/4' 9-1/4' - 9-3/4'	 4 3	 52 47	Water • Difficult driving • Unable to maintain plumb • Lower 6" of pile was completely destroyed (torn and bent) • Unable to drive below 9-3/4'

Notes:
[1]See Figure 5 for sheet pile driving test location.
[2]Vibratory hammer types used: a. Midsize: Tunkers 60.05 rated at 66 tons drive force; b. Large size: MKT V-30 rated at 160 tons drive force.
[3]Pile type: Pair of 3/8" wall thickness, Z sheets with pile tips.
[4]At locations B-1 (2nd attempt), B-5 and B-6, the sheet piles were driven after the uppermost 5 - 6 feet of riverbed materials were removed using an excavator.

APPENDIX D

GEOPHYSICAL SURVEY REPORT

SIGMUND D. SCHWARZ
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September 25, 1990
S86-90R

GeoEngineers, Inc.
2405 149th Avenue N.E.
Suite 105
Bellevue, Washington 98005

Att: Gordon Denby, PE

Re: Report of Geophysical Surveys, Puget Power White River Dam Reconstruction Project, Buckley, Washington

SUMMARY

Results of several geophysical surveys completed at this site indicate the area to be underlain by dense, relatively coarse grained alluvial and mudflow deposits. These deposits are indicated by their geophysical properties to be generally of uniform characteristics within the area explored and typical of the materials identified by the test borings. Bedrock occurs at relatively shallow depth several hundred feet north of the damsite area and deepens beyond the depth of exploration to the south beneath the river.

The primary objective of this work has been to characterize the nature of materials underlying the existing White River Dam to assist in the geotechnical aspects of design for the new structure and more specifically to identify anomalous zones where unexpected conditions might be encountered. The north abutment area has also been studied to develop information concerning potential leakage paths.

Geophysical surveys incorporating several complementary exploration methods have been completed at this site. Some elements of this work have been carried out under subcontract or by previous contract. These methods include seismic, electrical and electromagnetic techniques that are listed as follows:

1-GPR(ground penetrating radar)	Williamson and Associates
2-EM(electromagnetic)	Williamson and Associates
3-VES(vertical electrical soundings)	Schwarz
4-Overwater seismic refraction	Schwarz
5-Land seismic refraction	EBASCO Services (Nov. 1982)

The location and results of this survey are shown on the Geophysical Exploration Plan, Fig. 1 and Composite Geophysical

Profile A-A', Fig. 2 which includes the interpreted result of all geophysical surveys.

The damsite area was explored with 6 VES soundings, GPR, EM and overwater seismic refraction. The overwater seismic refraction survey was confused by the presence of high velocity concrete in the foundation of the existing dam and was therefore ineffective.

The north abutment area was explored by GPR and EM methods together with a land seismic refraction survey completed by EBASCO Services for Puget Sound Power and Light Corporation in 1982.

Interpreted results of the geophysical surveys are shown on the Composite Geophysical Profile A-A', Fig. 2. VES and GPR data are the most effective for delineating overburden stratigraphy in the damsite area. The VES data is depicted on the profile as a matrix of calculated electrical resistivity values derived from the six VES soundings and expressed as electrical resistivity in terms of ohm metres. Over this is superimposed GPR reflecting boundaries and the average seismic velocity. The GPR and EM survey was extended into the north abutment area to supplement the seismic refraction data presented in the EBASCO report. The agreement between data obtained by various geophysical methods and the borings is generally good.

Based upon these data, it appears that the site area is underlain by a fairly thin mantle of coarse grained recent alluvium over volcanic mudflow. The mudflow deposit is indicated by the seismic, VES and GPR data to be dense, basically hetrogeneous and very crudely stratified with a gentle southerly dip. The electrical and seismic characteristics of the mudflow are typical of those observed in the Osceola at other locations in the area.

The EBASCO seismic survey indicates bedrock to occur within 25 to 30 feet of the ground surface near the far north end of the geophysical profile and to be deepening to the south. These data indicate bedrock to be deeper than 60 to 70 feet in the damsite area.

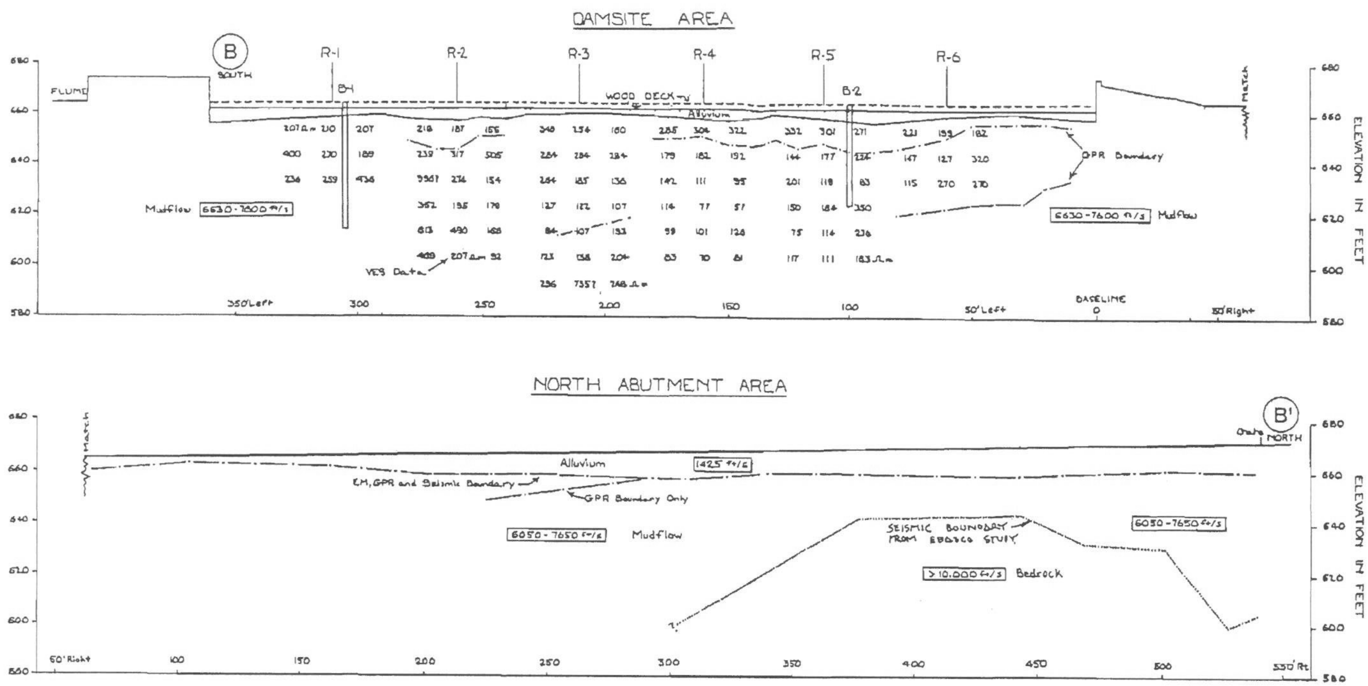
The data presented herein is interpretive in nature and subject to revision on the basis of additional site specific information that may become available.

Please do not hesitate to contact me if you have any questions concerning this report or if I may be of further service.

Respectfully submitted,

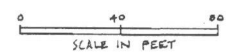
Sigmund D. Schwarz

Encl: Fig. 1 Exploration Plan, Fig. 2 Composite Profile A-A'



EXPLANATION:

GPR	GROUND PENETRATING RADAR
R-1	VERTICAL ELECTRICAL SOUNDING (VES) LOCATION AND NUMBER
284	VES RESITIVITY DATA IN MICRO OHMS
B-1,2	GEI BORINGS (THIS STUDY, REFER TO FIGURE 3 AND APPENDIX A)
------	EBASCO STUDY (1982) SEISMIC BOUNDARY
1425 ft/s	CALCULATED VELOCITY FROM SEISMIC STUDIES



NOTE: The information presented herein is interpretive in nature and subject to revision on the basis of additional site-specific information that may become available.

REFER TO FIGURE 1 FOR PROFILE LOCATION

WHITE RIVER DAM

COMPOSITE GEOPHYSICAL PROFILE B-B'

886-80

S.D. Schwarz, Consultant 9-25-90

REVISED BY GEOENGINEERS 10/12/90

FIGURE B-1

D-2, Seismic Ground Motion Evaluation

Mud Mountain Dam Fish Passage
Seismic Hazard Review

3 August 2004

Summary In accordance with ER 1110-2-1806, Design earthquakes and ground motions for the Mud Mountain Dam Fish Passage and Diversion Structures is presented. A peak ground acceleration of 0.34g is predicted at this site for the maximum credible earthquake (MCE) from a magnitude 9 event on the Cascadia Subduction Zone (CSZ). The peak ground acceleration for an Operating Basis Earthquake is 0.14 g. This represents an event with a 50% probability of occurring during the 100-year service life of the structures.

Project Description A diversion structure is located in the City of Buckley, 6 miles downstream of Mud Mountain Dam (MMD). The structure was constructed by a private entity in the early 1900's, to divert water into Lake Tapps for the White River Hydroelectric Project. The structure is a 352-foot-long, 11-fooot-high, timber crib dam with a concrete intake that diverts water from the White River at Buckley into a flume. When Mud Mountain Dam was built, in the WWII era, the fish collection facility was built at the diversion structure to provide fish passage around MMD. Since then, it has operated to provide water to the hydroelectric project and to direct upstream-migrating fish into the fish collection facility. The Muckleshoot Tribe built a hatchery on the right bank of the White River at the structure.

In the 1990's, PSE proposed construction and operation of a new power generating facility that would increase generation capacity, and had to obtain a FERC license in order to proceed with the modifications. Conditions required by FERC for licensing were deemed economically infeasible. PSE has decided to cease operations of the Hydroelectric Project. The Corps has entered into an interim agreement with PSE, where the Corps contracts with PSE to continue to operate the diversion structure.

Since the diversion structure has surpassed its economic life and is at risk of failure, Congress gave the Corps authorization to renovate the fish passage facility, including the diversion structure. The Army Corps of Engineers is currently reviewing a 35% design for both a federally preferred plan that meets fish passage objectives and a locally preferred plan, which provides for fish passage and diversion to Lake Tapps.

Geology

The Puget Lowland is an elongate topographic and structural depression between the Cascade Mountains on the east and the Olympic Mountains to the west. The structural depression that forms the Lowland consists of a series of basins (Tacoma, Seattle, and Everett) that are underlain by Tertiary volcanic and sedimentary rock. The Quaternary sediments, which unconformably overlie the Tertiary bedrock and fill the basins, are up to 3,600 feet thick.

The fish passage/diversion structures are located in a geologic region characterized by thin alluvial deposits overlying Osceola Mudflow deposits up to 70 feet thick in some areas. The Osceola Mudflow blanketed an extensive portion of the eastern Puget Sound Lowland approximately 3,700 years ago. The mudflow originated on the north flank of Mount Rainier, flowed down the White River Valley, and covered a wide area including present day Buckley with several tens of feet of sediment. The mudflow sediment typically consists of cobbly, silty sand and gravel with cobbles and occasional boulders. The mudflow occurred in a series of separate flows in between and after which alluvial soils consisting of sand and gravel with cobbles and boulders were deposited by the White River. Moreover, water flow over and through the surface layer of the exposed mudflow deposits probably washed silt material portions out of the soil. This process resulted in a relatively inhomogeneous mix of clean alluvial and mudflow deposits, as well as siltier mudflow deposits in the upper 5 to 10 feet of soil. Glacially deposited sands and gravels typically underlie the mudflow deposits.

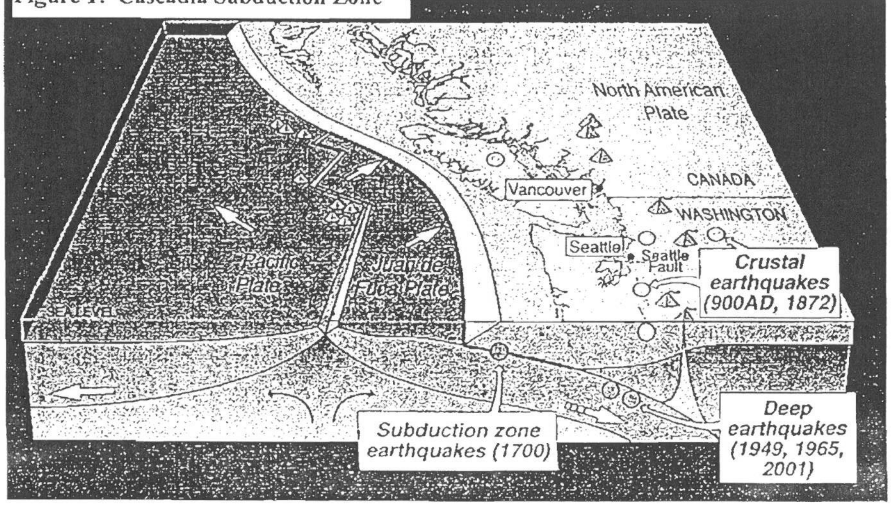
Soils exposed with a 25-foot-high riverbank along the south side of the White River, immediately upstream from the structures, show approximately 10 feet of an alluvial deposit overlying Osceola Mudflow sediments, indicating that the river has and continues to incise through the mudflow deposits at this location.

Tectonics

The tectonics and seismicity of western Washington is dominated by the Cascadia Subduction Zone (CSZ) giving rise to potential seismic sources that are generally divided into three categories: crustal, intraslab, and interplate.

The CSZ is an active subduction zone off the western coast of North America that extends over a length of 700 miles from southern British Columbia in the north to northern California in the south (Figure 1). Over most of the CSZ, the Juan de Fuca plate is subducting beneath the North American plate. The plates converge in a generally northeasterly direction at a rate of 2 to 4

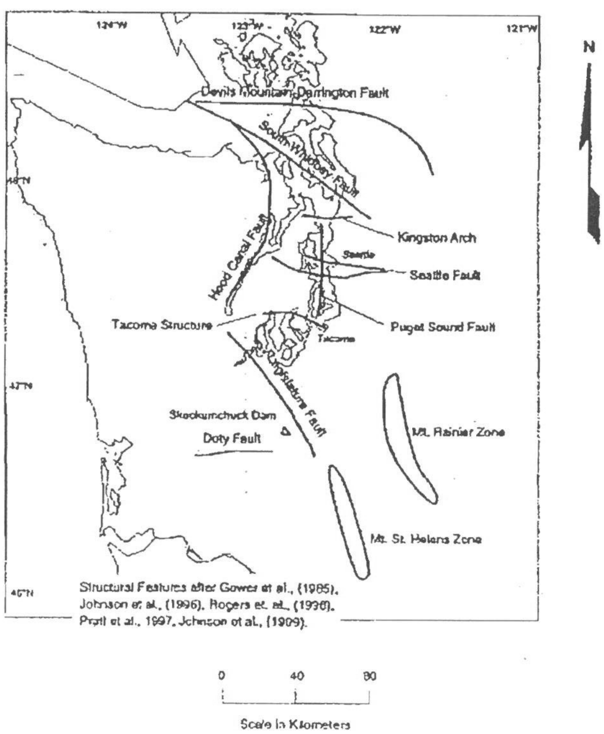
Figure 1: Cascadia Subduction Zone



centimeters per year. Subduction zones can produce thrust events on the interface between the subducting and overriding plates. Such interplate earthquakes can release large amounts of energy. The lack of observed interplate earthquakes on the CSZ raises questions about its potential for producing large magnitude events. This behavior can alternatively be interpreted as characteristic of weak coupling between the plates that allows convergence to take place continuously (and aseismically), or as a quiet period in which strain energy is accumulating in a locked zone between the occurrences of large earthquakes. Earthquakes can also originate within the subducting plate. Such intraslab earthquakes are extensional events that occur within the subducting Juan de Fuca plate. As the Juan de Fuca plate subducts beneath the North American plate, stress and physical changes in the subducting plate produce high-angle normal faulting earthquakes such as the 1949 Olympia, 1965 Seattle-Tacoma, and the 2001 Nisqually events. Figure 1 shows a cross section that identifies these earthquake sources through the central Puget Sound Basin, based on Hyndman and Wang (1995) and Stanley et al., (1999).

Both areal source zones and discrete faults are used to characterize crustal sources. Areal source zones are used to model much of the crustal seismogenic potential because evidence of bedrock structure of most of the Puget Lowland is concealed by thick Quaternary deposits and repeated glaciation. Crustal faults identified within the Puget Lowland with evidence of Late Pleistocene or Holocene (e.g., Seattle Fault, Puget Sound Fault) are considered discrete sources. The Mount St. Helens and Mount Rainer zones are also considered as discrete zones. There is substantial evidence for Quaternary movement on structures with the Puget Sound Basin.

Figure 2: Crustal Faults in Western Washington



While the bedrock structure of the Puget Sound Basin terrain is largely concealed by thick Quaternary deposits and repeated glaciation, it has been the subject of recent and on-going scientific research in the area. Faults and structures in and adjacent to western Washington are shown on Figure 2. This on-going research suggests that the north-south compression of the this terrain is being accommodated primarily beneath the Puget Lowland by a series of west and northwest trending faults or structures that extend to a depth of about 14 to 20 kilometers. These structures extend from the Doty Fault near Chehalis, north to the Darrington-Devils Mountain Fault near

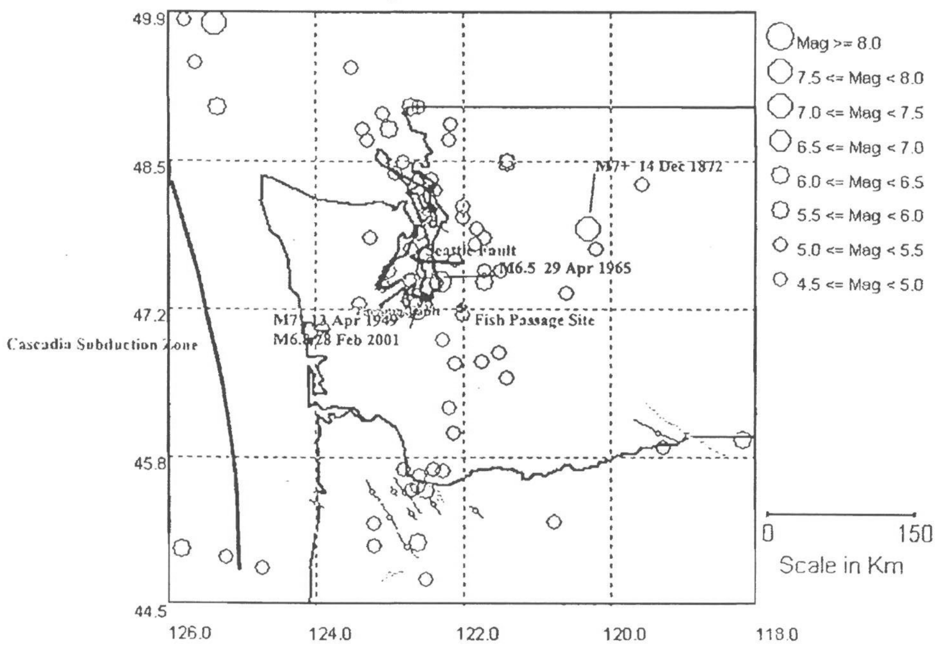
Anacortes and include the Tacoma structure and the Seattle Fault. However, geologic or geophysical evidence of Holocene movement has only been observed to date for the Seattle Fault.

Seismicity

The project site is located in a moderately active tectonic region that has been subjected to numerous earthquakes of low to moderate strength and occasionally to strong shocks during the brief 170-year historical record in the Pacific Northwest. Prior to the 1940's, historical events were primarily recorded using the Modified Mercalli intensity scale. Since the 1940s, earthquakes have generally been reported using magnitude scales. Earthquake magnitudes may correspond to different scales, including surface waves (M_S) body waves (m_b), Richter local magnitude (M_L), and moment magnitude (M_W).

The largest historic earthquakes to affect the site include the magnitude (M_S) 7.1 Olympia earthquake of April 13, 1949, the magnitude (m_b) 6.5 Seattle-Tacoma earthquake of April 29, 1965, and the February 28, 2001, magnitude (M_W) 6.8 Nisqually earthquake. These events were

Figure 3: M4.5 Earthquakes or Greater From 1977 to 2001



located (epicentral distance) approximately 51 miles (1949 & 2001) east and 55 miles (1965) southeast of the project site. Ground shaking in the Buckley area near the site was reported as Modified Mercalli intensity VII to VIII (1949) and VII (1965), and as peak ground acceleration of 0.1g (2001). The 1949, 1965 and 2001 events were located in the subducted Juan de Fuca slab beneath the Lowland at depths of 33 to 39 miles. The level of ground shaking that occurred during these events at the project site is likely the maximum vibratory ground motion that would have occurred at the project site during the 170 years of historical record.

Other large historic earthquakes felt in western Washington include the 1872 North Cascades earthquake and two other events in western British Columbia, Canada. The North Cascades earthquake of December 15, 1872, appears to have been one of the largest crustal earthquakes in the Pacific Northwest, with an estimated magnitude of 7+ and a maximum intensity of VIII. Although the epicentral location of this event is uncertain, owing to the sparse population of the area at that time, it apparently was a shallow crustal event located about 100 miles (epicentral distance) northeast of the project site somewhere in the north Cascades-Okanogan region. In Canada, major earthquakes occurred on Vancouver Island on June 23, 1946, and in the Queen Charlotte Islands on August 21, 1949 (Coffman and von Hake, 1973). These events had magnitudes of 7.3 and 8.1, respectively. Because of the large distance of these earthquakes from the project site (over 150 miles), there were no reports of significant damage in the area.

Seismic Hazard Evaluation

The assessment of the MMD Fish Passage and diversion structures seismic hazards is based on existing seismic hazard data and studies that represent the state-of-the-knowledge and -practice for this region.

Site Classification

The project site has a Low Hazard Potential Classification in accordance with Appendix B of ER 1110-2-1806. The critical feature of the project is proposed to be a concrete embankment, concrete fish passage facility and earthen levees.

Design Earthquakes

Development of maximum credible earthquake (MCE) ground motions is based on deterministic seismic hazard analyses (DSHA). An MCE can be defined as the largest likely ground motions that may occur at a site from a capable seismogenic source. The basic inputs to a deterministic analysis are the fault type, maximum magnitude, the distance from the nearest point of the fault plane to the project site, and the ground motion attenuation behavior. MCEs are selected based on the largest ground motions that may occur at the site from capable seismogenic sources. Two MCE sources were determined to be controlling faults of the project site: the Cascadia Subduction Zone Interplate and the Seattle Fault.

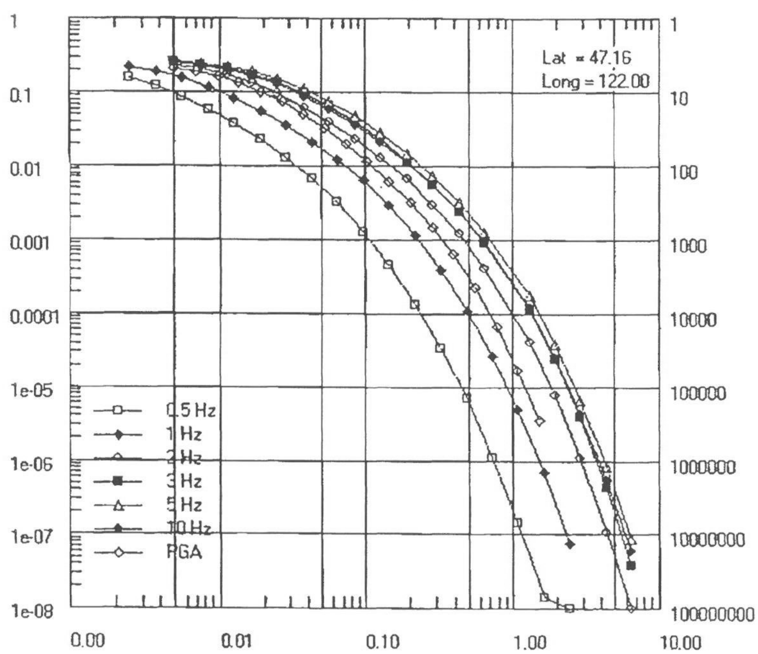
The maximum magnitude associated with the Cascadia Subduction Zone Interplate is M_w 9.0, which requires rupture of nearly the entire subduction zone. The closest approach of the seismogenic rupture approaches to within 81 kilometers of the site. Horizontal peak ground acceleration and response spectra were estimated for the site using the empirical attenuation relationship of Youngs et al. (1997).

Table 1: Source Zones and Magnitudes

	Magnitude	Distance (km)	Peak Horizontal Ground Acceleration (g)			
			Horizontal		Vertical	
			Mean	+ 1σ	Mean	+ 1σ
CSZ	9	81	.34	.63	n/a	n/a
Seattle Fault	7.2	19	.32	.57	.29	.43
Tacoma Fault	6.9	34	.14	.21	.07	.10

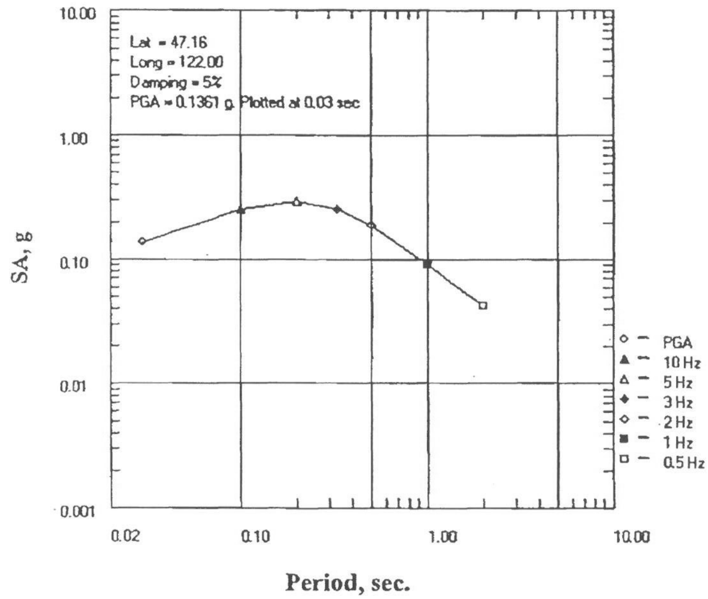
According to ER 1110-2-1806, an Operating Basis Earthquake (OBE) is based on the event with a 50% probability of occurring during the 100-year service life of the structures. This translates to a 144-year return period. The OBE is used to design against economic losses. The probabilistic hazard characterization is based on existing USGS data developed for the NEHRP, NSHMP (Frankel et al, 1996). The set of hazard curves for the Fish Passage site are for PGA and spectral frequencies of 0.5, 1, 2, 3, 5, and 10 Hz, which correspond to periods of 2.0, 1.0, 0.5, 0.3, 0.2 and 0.1 seconds (Figure 4). These curves are the basis for equal hazard response spectra that are formed by selecting the points from each hazard curve for a given return period. The OBE spectral acceleration for return periods of 144 is presented in Figure 5. The peak ground acceleration for an OBE is 0.14 g.

Figure 4: PGA and SA Probabilistic Hazard Curves



Note: frequency of exceedence and return period (yrs) plotted on Y-axis. Acceleration in %g plotted on X-axis.

Figure 5: Equal Hazard Response Spectra for Operation Bases Earthquake (144 year return)



References

Abrahamson, N.A. and Silva, W.J. (1997). "Empirical Response Spectral Attenuation Relations for Shallow Crustal Earthquakes," *Seismological Research Letters*, v. 68, No. 1, January/February, p. 94-127.

Frankel A. D., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E. V., Hanson, S., and Hopper, M. (1996). A National Seismic Hazard Maps, June 1996 Documentation, Open-File Report 96-532, U. S. Geologic Survey, Golden, CO.

Gower, H.D., Yount, J.D., and Crosson, R.S. (1985). "Seismotectonic Map of the Puget Sound Region, Washington," U.S. Geological Survey Miscellaneous Investigations Series Map I- 1613, scale 1:250,000.

Johnson, S.Y., Potter, C.J., Armentrout, J.M., and others, (1996). "The Southern Whidbey Island Fault, an Active Structure in the Puget Lowland, Washington," *Geological Society of America Bulletin*, v. 108, p. 334-354.

Johnson, S.Y., Dadisman, S.V., Childs, J.R., and Stanley, W.D. (1999). "Active Tectonics of the Seattle Fault and Central Puget Sound, Washington--Implications for Earthquake Hazards," *Geological Society of America Bulletin*, v. 111, No. 7, p. 1042-1053, July.

Pratt, T.L., Johnson, S.Y., Potter, C.J., and others, (1997). "Seismic-reflection Images Beneath Puget Sound, Western Washington State," *Journal of Geophysical Research*, v. 102, p. 469-490.

Rogers, A.M., Walsh, T.J., Kockelman, W.J., and Priest, G.R. (1996). "Earthquake Hazards in the Pacific Northwest--An Overview," *in* A.M. Rogers, T.J. Walsh, W.J. Kockelman and G.R. Priest (eds.), Assessing Earthquake Hazards and Reducing Risk in the Pacific Northwest, U.S. Geological Survey Professional Paper 1560, v. 1, p. 1-54.

Youngs, R.R., Chiou, S.-J., Silva, W.J., and Humphrey, J.R. (1997). "Strong Ground Motion Attenuation Relationships for Subduction Zone Earthquakes," *Seismological Research Letters*, v. 68, No. 1, January/February, p. 58-73.

U.S. Army Corps of Engineers (1995). ER-1110-2-1806, Earthquake Design and Evaluation For Civil Works Projects.

MWH
MONTGOMERY WATSON HARZA

EMPLOYEE TRAVEL AND EXPENSE STATEMENT

Page 1 of 1

Employee Name: ____________ **Employee No:** ____________ **Week Ending:** ____________ **Home Bus. Unit:** ________ **Location:** ________

Ref.	Date	Time Start	Time End	Job Number	Cost Code	Type D	Type O	Type T	Type M	Personal Auto Miles	Personal Auto $	Meals	Other Travel Costs Trans	Other Travel Costs Lodging	Other Travel Costs Misc.	SUBTOTAL	Entertainment $	Other Expenses $	Other Expenses Cost Type	TOTAL EXPENSES	Acctg. Use Only Unallow. Costs
1											-					-				-	
2											-					-				-	
3											-					-				-	
4											-					-				-	
5											-					-				-	
6											-					-				-	
7											-					-				-	
8											-					-				-	
9											-					-				-	
10											-					-				-	
11											-					-				-	
12											-					-				-	
				TOTALS							-	-				-	-	-		-	
				Cost Type - Direct		D					5885	5665				5655	-		Various		5620
				Cost Type - Overhead			O				6710	6665				6655	6665		Various		6655
				Cost Type - Training				T			6710	6665				6655	-		Various		6655
				Cost Type - Marketing					M		7710	7665				7655	7665		Various		7657

Explanation of business purpose for expenses as shown on above lines are to be provided below. Original receipts are required for all expenses greater than $25. Explanation of business meals and meetings are to include details supporting business purpose and names of guests. Attach additional pages as required.

1 ____________ 7 ____________
2 ____________ 8 ____________
3 ____________ 9 ____________
4 ____________ 10 ____________
5 ____________ 11 ____________
6 ____________ 12 ____________

Employee Signature ____________ Date ____________

Approval Signature ____________ Date ____________